SCHOOL OF CIVIL ENGINEERING

INDIANA DEPARTMENT OF HIGHWAYS

JOINT HIGHWAY RESEARCH PROJECT

FHWA/IN/JHRP-83/14

IMPROVING EMBANKMENT DESIGN AND PERFORMANCE

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PURDUE UNIVERSITY

JOINT HIGHWAY RESEARCH PROJECT

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Final Report

"Improving Embankment Design and Performance"

To: Harold L. Michael, Director
Joint Highway Research Project

October 12, 1983
Revised December 1983
Project: C-36-5M

From: A. G. Altschaeffl, Research Engineer

File: 6-6-13

Please find attached the Final Report on the HPR Part II Study entitled, "Improving Embankment Design and Performance". The report has been prepared by A. G. Altschaeffl and C. W. Lovell of our staff.

The results of the study have fulfilled the objective to improve the engineer's capability to predict the behavior of field compacted soil. Procedures, and accompanying usable charts and diagrams, are presented that allow: (1) the creation of a specification for compaction that will assure a desired field behavior parameter; (2) the prediction of field behavior parameters from only field inspection testing results. These represent a significant addition to the state-of-the-art.

In all cases of behavior parameters the magnitudes of the dry density and water content on the as-compacted lift must be controlled or known. In addition, the range of water content plays a major role in the variability of the parameters. These findings indicate a strong need to re-examine Indiana's compaction specifications and to change them to include the regulation of dry density and water content for field compaction; only in this way can there be assured control of behavior in-service, the prerequisite to minimizing the need for premature maintenance.

We also believe the results of this study for 2 soils are of such promise that an enlargement of the data base should be initiated at an early date and prosecuted in a planned manner. A plan is presented herein.

This Final Report is submitted for review and approval as fulfillment of the objectives of this project.

Respectfully,

A. G. Altschaeffl /ms

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FINAL REPORT

"Improving Embankment Design and Performance"

by

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Joint Highway Research Project

Project No.: C-36-5M

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16. Abstract

This project involved compacted soil and the prediction of its behavior. Two soils were examined (A-4, A-7-6), with 2 rollers each; 2 test pads were included in the methodical examination of behavior. Exhaustive testing was performed of both laboratory and field compacted conditions.

Procedures were created that allow: 1) creation of a design specification for compaction that will assure a desired field behavior parameter; 2) prediction of the field parameters knowing only inspection testing results; 3) accounting for variability in parameters in design analyses. These procedures are significant additions to the state-of-the-art.

In all cases of parameters, magnitudes of water content and dry density, as-compacted, must be controlled or known. The range of water content, as-compacted, plays a major role in property variability. These findings led to recommendation for a re-examination of Indiana specifications to provoke more thorough regulation of dry density and water content; only in this way can there be assured control of behavior in-service, the pre-requisite to minimizing the need for premature maintenance.

An enlargement of the data base is recommended by an implementation program.

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Highlight Summary

This is the Final Report for this project. It represents an assembly of the work of the various tasks, as well as a compilation of charts and diagrams that can be directly useful to the soils engineer for earthwork design and performance.

This project was motivated by the need to improve the engineer's capability to predict in-service behavior of field compacted soils while accounting for the existing variability. The focus was intended to be upon the most difficult question facing the engineer in earthwork: "how much compaction should be specified"?

A methodical examination of construction records and several on-going construction projects led to the creation of 2 test pads and associated laboratory testing; only in this way, it was believed, could the variables controlling the field properties be properly evaluated. Two soils, an A-4 and an A-6 to A-7-6, and 2 rollers for each soil, were studied. Exhaustive testing of both laboratory and field compacted conditions was performed. The results of testing were evaluated and manipulated with statistical techniques; many charts and diagrams were created.

Procedures were created that allow: (1) the creation of a design specification for compaction that will assure a desired field behavior parameter for borrow known in advance of construction; (2) the prediction of the field behavior parameters from only inspection testing results when borrow is not known in

advance of construction. In addition, techniques are also presented to allow the use of behavior parameters in design analysis studies while accounting for the variabilities in the parameters. Several charts are presented to allow ready use of the results of this study's work. These procedures represent significant additions to the state-of-the-art.

In all cases of the behavior parameters, the magnitudes of the water content and dry density at the time of compaction must be controlled or known. In addition, the range of water content on the as-compacted lift plays a major role in the variability of the parameters. These findings indicate a strong need for Indiana to re-examine its earthwork specifications and to change them to include the regulation of dry density and water content for field compaction. Only in this way can there be assured definition and control of soil behavior in-service; this control is necessary if premature maintenance is to be minimized in the correction of landslides, sloughing, excessive settlement or heave of pavements, etc.

Finally, it is believed that the reported procedures are of such promise that an enlargement of the data base should be initiated at an early date and prosecuted in a planned manner.

Summary and Conclusions

Two soils (an A-4, and an A-6 to A-7-6) were thoroughly tested in field and in laboratory compacted conditions. Procedures have been developed that allow: (1) the creation of a design specification for compaction that will assure a desired field behavior parameter for borrow known in advance of construction; (2) the prediction of the behavior parameters from only inspection testing results when borrow is not known in advance of construction. Both of these developments represent significant additions to the state-of-the-art.

In order to make fullest use of these procedures for more efficient earthwork and improved performance in-service, several matters must be recognized. In all cases of the behavior parameters the magnitudes of water content and dry density at the time of compaction must be controlled. In addition, the quality of the prediction of behavior appears to be strongly related to the range of water content on the as-compacted lift; positive control of this water content range is a requirement for best earthwork.

For Indiana, it is strongly urged that consideration be given to a close examination of earthwork specifications and to change them to include the regulation of dry density and water content for field compaction. Without such regulation in-service behavior cannot be controlled effectively, and this leads to the need for premature maintenance to correct landslides, sloughing, excessive settlement or heave of pavements, etc.

The data base, and charts and tables therefrom, are limited to the soils and equipment examined in this study. It is believed that the procedures recommended in this report are of such promise for use that an enlargement of the data base is definitely appropriate; implementation of this recommendation is proposed.

Much has been learned about the behavior of compacted clays. The statistical models formed are of value in themselves, and form the base for a future simplified phenomenological model for compacted behavior.

Introduction

This report represents a summary of the work done by the various researchers associated with this project. It attempts to show that the project, indeed, fulfilled its objectives.

This project was motivated by the wish, and need, to improve the engineer's capability to predict the in-service behavior of field compacted soils. The variability in the field material was to be included in consideration.

The project was proposed in 1973. At that time, and to a large extent today, the engineer relied largely upon results of laboratory testing to provide the parameters he would use for his analyses of the field project. This was done with the awareness that the field behavior likely would differ from that of the same soil in the laboratory. It was infeasible to consider field trials, job by job, in the highway industry as a way to overcome the problem. This project was motivated as a methodical effort to improve the engineer's ability to predict field behavior.

In soils engineering, particularly for the commonly used clayey soils, it is known that the soil fabric controls the engineering response of the soil. This fabric, i.e., the arrangement and distribution of the constituent particles and pore sizes, is determined by the water content at the time of compaction and the manner in which the soil is placed. The field behavior is, thus, controlled by a fabric that will differ from that in the laboratory because compaction conditions are

different. The awareness of this fabric effect appeared to have caused the creation of rules of thumb or guidelines for the engineer. For example, it was said that a clay compacted on the dry side of the optimum water content would lose significant strength from its as-compacted state when the soil became wetted. It was such guidelines that the engineer used to create his specification for field compaction to create, he hoped, the pattern of field behavior he desired.

There are 3 questions that are addressed by the engineer in soil compaction: (1) how much compaction?; (2) how to achieve it?; (3) how to control it? The third question is the best answered, with many techniques available to measure in-place density and water content with which to compare with specified values. The second question also has generated responses with guidelines available, e.g., clays should be compacted with "kneading" type rollers. The first question, although the most important, is the one about which the least is available. There was no quantitative approach available to the engineer in 1974 to help him decide the contents of the compaction specification to assure him of his desired pattern of field behavior in his compacted soil. This was the arena to which this project hoped to contribute.

This project began with a focus on better prediction of field behavior including the effects of variability, as well as how to include these in design analyses. As the project evolved, the focus changed somewhat towards a better basis for preparing

the specification. The development of this evolution is presented in the next section wherein project history is presented. Following this is presented a summary of the various tasks performed in the project.

The findings of most of the tasks have been separated into 2 categories: 1) design engineering-creating a specification to assure a behavior, when borrow soil is identified in advance of construction; (2) quality assurance - to generate a prediction of properties from inspection test results, when the soil is not identified prior to construction. Charts are presented, for use, for the soils and equipment associated with the project. This major section is intended for practical use, as is, by the engineer to allow better prediction of behavior and an improved basis for creating the specification.

There follows a section which shows how well able the findings can predict behavior. Field projects were sampled and tested as an effort to validate the techniques created by the project.

The final portion of the body of the text contains some generalizations and conclusions based upon the work done for the soils and equipment tested. Suggestions and recommendations are made relative to procedures and specifications presently used by IDOH in earthwork. In addition, an implementation plan is proposed to enlarge the data base created by the project, to allow improved engineering with other soils and equipment.

Several appendices are presented which concern useful details about the procedures used in the project so others may use them as well.

Background and Project Chronology

The original motivation for the project was to try to create better predictability of field behavior of Indiana soils compacted in the field, while accounting for the variability which exists. A proposal was prepared and transmitted on September 14, 1973. This proposal included a Phase I-A concerning an assembly of compaction data and field behavior data, especially from the files of IDOH; it was thought possible that, with some additional testing of sampled field compacted soils, relations between laboratory and field data would present themselves. A Phase 1-B concerned the variability of the behavior parameters. A Phase II concerned how variability could be inserted into analyses. This proposal was approved for funding beginning January 1, 1974 (under letter dated December 27, 1973 from Mr. T. J. Ptak, FHWA) with a completion date of June 30, 1977.

An Interim Report was approved June 4, 1976 for portions of Phase I, as entitled: "Improving Embankment Design and Performance: Prediction of As-Compacted Field Strength by Laboratory Simulation", by J. L. Peterson (Report No. JHRP 75-22, December 1975).

An Interim Report was approved April 6, 1977 for portions of Phase I, as entitled: "An Examination of the Variability Resulting from Soil Compaction", by M. F. Essigmann, Jr. (Report No. JHRP 76-28, October, 1976).

An Interim Report was approved August 18, 1977, for portions of Phase I, as entitled: "Examination of the Variability of the Soaked Strength of a Laboratory Clay", by J. C. Scott (Report No. JHRP 77-8, May, 1977).

The foregoing tasks found there was much variability in behavior. The focus narrowed to the A-4 soil category (clay of low plasticity), and an on-going construction project was sampled for field data. The continued presence of large variability caused the project Advisory Committee to recommend a test embankment that was built and costed as part of an IDOH construction project. The continued presence of large variability in the test embankment suggested that it may be "normal". The Advisory Committee then recommended another, more clayey, soil be given attention. The work of the three Phase I tasks resulted in the determination that laboratory and field behavior relations were similar; this allowed the development of a concept for creating a specification that would assure a specific response behavior. Only strength was included in this development, but the results offered much promise for the enhancement of the design engineer's capability.

In order to allow fuller development of the concept to another soil and to additional behavior properties, a proposal was prepared for an extension of the project. The proposal was presented April 4, 1977. A meeting was held on June 7, 1977 with representatives of IDOH and FHWA to discuss the proposal and answer questions. Approval was received effective July 1, 1977

(under letter dated 12 July 1977 from Mr. M. J. Monahan, FHWA) for tasks as follows: 1) Group I - Clay of Low Plasticity - task EE; 2) Group II - Clay of Higher Plasticity - tasks A, B, C, D, E, FF, GG. A group of tasks concerning applications to practice were deferred until a later review could be made. Completion for authorized tasks was to be December 1980.

An Interim Report was approved September 14, 1978 for Task EE, as entitled: "Soil Compaction Specification Procedure for Desired Field Strength Response", by J. T. Price (Report No. JHRP 78-7), June 1978).

An Interim Report was approved October 19, 1979 for Task A, as entitled: "The Effect of Laboratory Compaction on the Compressibility of a Highly Plastic Clay", by A. DiBernardo and C. W. Lovell (Report No. FHWA/IN/JHRP-79/3, May 1979).

An Interim Report was approved December 11, 1979 for Task B, as entitled: "The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation", by J. M. Johnson and C. W. Lovell (Report No. FHWA/IN/JHRP-79/7, August 1979).

An Interim Report was approved January 9, 1980 for Task C, as entitled: "The Effect of Laboratory Compaction on the Unconsolidated-Undrained Strength Behavior of a Highly Plastic Clay", by D. W. Weitzel and C. W. Lovell (Report No. FHWA/IN/JHRP-79/11, September 1979).

An FHWA review was made of progress at a meeting with Purdue and IDOH personnel on September 17, 1979. This review indicated that progress was acceptable, that lack of personnel has caused some delay in completing tasks as scheduled originally, that work continue as has been authorized, and that a letter should be prepared to indicate researcher's plans beyond completion of ongoing tasks. Such a letter was submitted on February 4, 1980 with a proposed revision in the Work Plan to: 1) combine Group III tasks F and G into one new Task FFF; 2) to slightly enlarge Task HH; and 3) create a new Task H, the development of the Final Report. Approval was received effective August 12, 1980 (under letter from Mr. C. E. Basner, FHWA) with project completion scheduled for December 31, 1983.

An Interim Report was approved May 18, 1981 for Tasks FF and GG, as entitled: "Prediction and Control of Field Swell Pressures of Compacted Medium Plastic Clay" by G. M. Terdich (Report No. FHWA/IN/JHRP-80/4, March 1981).

An Interim Report was approved October 20, 1981 for Task D, as entitled: "Compressibility of Field Compacted Clay", by P. S. Lin and C. W. Lovell (Report No. FHWA/IN/JHRP-81/14, August 1981).

An Interim Report was approved March 26, 1982 for Task E, as entitled: "Strength of Field Compacted Clayey Embankments", by Y. Liang and C. W. Lovell (Report No. FHWA/IN/JHRP-82/1, February 1982).

The constraints of the 1974 budget became too large for completion and a request for a supplemental budget increase was presented April 16, 1982. Approval was received April 27, 1982 (under letter from Mr. C. E. Basner, FHWA).

An Interim Report was presented August 30, 1983 for Task FFF, as entitled: "Design of Compacted Clay Embankments for Improved Stability and Settlement Performance", by M. J. Goodman, J. L. Chameau and C. W. Lovell (Report No. FHWA/IN/JHRP- , August 1983).

The results of Task HH work have been made part of this Final Report of the project.

Task Summaries

Phase I - Improving Predictability of Field Behavior

Background

The soil compaction process produces a soil mass and fabric that differ from that of the laboratory. The process also produces a mass with variability in its characteristics. A better understanding of these differences was believed the key to better prediction by the engineer of field behavior parameters. This study postulated there were functional relationships between property parameters and the compaction variables for the laboratory and for the field, and a correlation between them should be possible. If this could be done, prediction of field parameters would be possible from results of laboratory testing. In addition, it was thought that the determination of variabilities in parameters could possibly lead to procedures allowing the engineer to custom-make specifications to assure presence of desired behavior in the field. There was the hope that available literature and the files of IDOH would contain many data to allow testing the hypothesis.

Investigation

The first study in this Phase I assembled data from published literature and from the files of IDOH. Published data did yield trends, as expected; construction data from IDOH did not. The small sample size of the former and the non-homogeneity of

the latter prevented conclusive inferences. Field samples were secured from construction of an IDOH embankment of A-4 soil. These were tested, and searches were made for functional relationships. Similar data were obtained from samples of the same soil compacted in the laboratory. Attempts were then made to predict field parameters from laboratory relations.

The second study focused on the effects of the variability of the strength of the A-4 soil. Laboratory data were generated and efforts were made to correlate with field parameters. Data were manipulated in an effort to suggest how to create specifications to assure the presence of a given parameter magnitude in the field.

The third study focused upon the soaked strength of the A-4 soil compacted in the laboratory. This was to determine the effects on variability of parameters allowing for a (simulating) environmental change. Again, data were manipulated to suggest the effect of soaking upon the procedures to create a specification for compaction.

Conclusions

Data collected from literature and construction records were inadequate to allow determination of the sources of variability. Field and laboratory data collected for the A-4 soil yielded relationships that were similar (but not identical) and with differing variabilities. An attempted prediction of field as-compacted strength was promising, but the need was clear to

better understand the causes of the variability. The data clearly showed that the dry density was a factor of far lesser importance in determining the strength than was the water content at the time of compaction. Water content control was deemed critical if strength is to be the design control for the field.

In the laboratory operation, the importance of the water content was reinforced as the major determinant of the parameter magnitudes. The variability in the parameter was determined by the magnitude of the parameter as well as by the water content. No simple one-on-one relations between the parameters and the compaction variable were found; significant relations usually contained interactions between, for example, water content and compactive energy.

Forecasting of field behavior continued to be poor; a recommendation was made for a test pad to allow better control and better determination of field parameters.

A procedure was suggested for creating the specification for compaction, and it was believed to show promise for use.

When soaking was involved, the normalized variation in soaked strength was found to be larger than that for as-compacted strength. The magnitude and variability of soaked strength on

the dry side of optimum water content differed from those on the wet side. Interaction terms again were important in the relations, and one-on-one relationships with compaction variables were not useful. The effects of these findings were related to the technique for creating field specification.

the dry side of optimum water content differed from those on the wet side. Interaction terms again were important in the relations, and one-on-one relationships with compaction variables were not useful. The effects of these findings were related to the technique for creating field specifications.

Task A - The Effect of Laboratory Compaction on the Compressibility of a Highly Plastic Clay

Background

Predicting the settlement of an embankment, due to movements within that embankment, is important for high embankments and/or embankment enlargements. This study affords empirical evidence and guidance for accomplishing this objective with highly plastic compacted clays. Compacted samples were loaded one dimensionally to determine the compactive prestress. Other samples were loaded to simulate a variety of embankment positions, and the 1-D volume changes produced by saturation were measured. Both the prestress and the volume change were peculiar to the details of the compaction process.

Experimentation

The soil was a highly plastic residual clay from shale. It was compacted to fit a variety of impact type moisture-density curves, but in a kneading compactor. The kneading compactive effort was varied by adjusting the foot pressure. The extra procedure of preparing compressibility samples by kneading compaction was justified on the bases of: (a) a better approximation of field compaction modes and total efforts, and (b) a lesser variability of moisture, density and perhaps fabric within a given sample.

Compressibility of the as-compacted material was determined by loading in a conventional oedometer. It was found that compression proceeded through the initial and primary stages and into the secondary or creep range within 10 minutes. It was thus possible to generate the load-volumetric strain relationship very quickly and avoid problems of moisture change in the samples. This relationship was interpreted via the conventional Casagrande construction to define the compactive prestress. Compressibility both below and above the prestress was also defined.

Other compacted samples were loaded to approximate various positions in a high embankment and were back-pressure saturated to produce an abrupt volume change of either heave or settlement. These movements modeled those which would occur in the real embankment when it became saturated in service. These same samples were additionally loaded to define the saturated compressibilities. Pore pressure dissipation measurements for the saturated samples showed that the log-time curve fitting procedure was a highly satisfactory technique for defining the amount of primary consolidation.

All experimental procedures for defining the compressibility parameters of highly plastic and laboratory compacted clays are described in detail in JHRP Report No. 79-3. These procedures can be recreated by competent interested parties through reference to JHRP 79-3.

Conclusions

In certain instances, embankment settlements due to movements within the embankment, need be predicted from laboratory measurements. For those cases, this study presents a model plan for testing and interpretation.

Compactive prestress is increased by (a) decreased compaction water content, and (b) increased compaction pressure, as shown in Figure 37[1], JHRP 79-3, for the test clay. This effect is not important for controlling settlements in the as-compacted condition, since these settlements will occur as rapidly as the embankment is constructed. Compactive prestress also influences the volume changes incurred on saturation, but this effect was not quantitatively defined in this research task.

Figure 38, JHRP 79-3 shows the prediction for volume change on saturation, with a decrease in settlement (collapse) caused by: (a) an increase in compaction water content at constant compaction pressure and (b) an increase in compaction pressure at constant compaction water content. This specific prediction equation holds only for the test soil within the zone of observation values (Figure 39, JHRP 79-3). However, the form of the functional relationship should hold for similar soils, and is instructive in the control of movements through proper selection of compaction specifications. The interpretations of this report may be closely followed for embankments to be constructed of other clays.

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[1] Cited illustrations are collected in Appendix F, pages 133-168.

Predictions developed for field compacted clays are covered in a later task.

Task B - The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation

Background

Embankments are usually assessed as to stability against shear failure at two times: (a) end of construction, and (b) in the long term. This task provides a model for accomplishing the latter evaluation, assuming that the embankment material becomes fully saturated under a confinement approximating various positions in said embankment. Laboratory compacted samples have thus been confined, back pressure saturated, and sheared undrained with pore pressure measurement. Statistical relations were developed between the variables in the compaction process and (a) volume changes on saturation, (b) the Mohr-Coulomb effective stress strength intercept (c'), (c) the Mohr-Coulomb effective stress strength angle (ϕ'), and (d) the Skempton pore pressure parameter, A_f.

Experimentation

The soil studied was a highly plastic residual clay from shale. It was kneading compacted to fit various impact-type compaction curves, and isotropically confined to approximate various positions in a high embankment. The samples were then back-pressure saturated and the volume changes were measured. Finally, the samples were failed in undrained shear with pore pressure measurements. The testing technology of this task is advanced, but well within the capability of a normal soil testing laboratory. The testing procedures are described in detail in JHRP 79-7, and can be repeated by other investigators through referral to them.

Conclusions

The results of this study may be used as a model for assessing the stability of a compacted high embankment from testing of laboratory compacted clay. The volume changes caused by saturation under confinement are found to be principal functions of compacted dry density, compacted degree of saturation, and the pressure under which saturated (equation 3-5, JHRP 79-7). This equation is probably less useful for predicting settlements of an embankment than the comparable equation in JHRP 79-3 (Figure 38), since it involves 3-D rather than 1-D volume change. However, a nomographic solution to the prediction equation is given as Figure 52, JHRP 79-7, and the region of values for which it is valid is shown as Figure 51, JHRP 79-7.

The prediction model for the effective stress strength intercept is given as equation 3.8, (JHRP 79-7), and is a function of compaction water content and compacted void ratio. The fact of primary importance here is the very small effect of these compaction variables on the value of c'. In a practical problem, it is suggested that c' may be taken as a single average value. The effective stress strength angle (ϕ') is essentially a constant for all reasonable values of the compaction variables. Thus, the long term <u>drained</u> strength of a laboratory compacted highly plastic clay is essentially independent of the details of compaction. See Figure 63, JHRP 79-7, for the joint region of observations.

Shearing-induced pore pressures <u>do</u> depend in an important way upon the compacted dry density, the compacted degree of saturation, and the confinement under which the material was saturated. This is shown in equation 3-6 (JHRP 79-7) and in the nomographic solution of Figure 59, JHRP 79-7. Thus the undrained strength in the long term <u>is</u> affected by the above listed compaction and position variables. Of particular interest to the engineer wishing to predict stability is the sign of the pore pressure at failure. Nearly all values of A_f for this clay are positive, denoting decreased effective stress, and reduced strength, with respect to a drained case.

Lacking data of the type discussed above for <u>field</u> compacted highly plastic clay, the procedures and interpretations of JHRP 79-7 present a highly usable model for determination of embankment stability. Regression coefficients will vary with the specific soil, but the functional forms of the equations are expected to be about the same. Interpretations do suffer somewhat from the lack of definition of compactive prestress and its modification by saturation and consequent volume change.

Task C - The Effect of Laboratory Compaction on the Unconsolidated-Undrained Strength Behavior of a Highly Plastic Clay

Background

Stability of a high compacted clay embankment is generally prredicted for two critical times in the service life, viz., (a) at the end of construction, and (b) in the long term. This task concentrated on the evaluation of a laboratory-compacted highly plastic clay for the end-of-construction circumstance. To accomplish this objective, samples were compacted, confined undrained, and sheared undrained. The undrained strengths, defined in terms of compaction and position variables, are the values which would ordinarily be applied in stability analysis of an embankment.

Experimentation

The soil was a highly plastic residual clay shale. It was compacted to fit various impact type moisture-density curves by varying the foot pressure of a California type kneading compactor. Compactive work was computed by integrating force-deformation relations for each tamp.

Compacted samples were isotropically confined to approximate positions in a high embankment and were sheared undrained. Statistical relationships were then developed to predict density, strength, and strength variability; all of these were expressed in the form of nomographs or graphs. All testing and analytical

procedures are thoroughly described in JHRP 79-11, and can be repeated in detail by refereence to this report.

Conclusions

Compacted density is affected by compactive energy on the dry side of the line of optimums, but unaffected on the wet side. Accordingly, it was necessary to form different prediction equations for each side of optimum. The prediction for dry side dry density is given by equation 3.8 (JHRP 79-11), and is dependent upon the compaction water content and the ratio of average compactive work by kneading compaction (Table 3.6, JHRP 79-11). The prediction for dry density wet-of-optimum is given by equation 3.9, JHRP 79-11, and is a function of compaction water content only. These equations strictly apply for the single test soil within the zone of observed values of independent variables.

As-compacted strength is also affected by compactive energy on the dry side of the line of optimums, but unaffected on the wet side. Accordingly, separate UU strength predictions are develooped. The prediction dry of optimum varies with the as-compacted dry density, the as-compacted degree of saturation and water content, and the confinement under which sheared (equation 3.11, JHRP 79-11). The wet-of-optimum strength is given by equation 3.15, JHRP 79-11, and varies only with the compacted void ratio. The variability of compacted strength is found to increase with the magnitude of strength (Figure 3.30, JHRP 79-

11). Minimum expected strength can be calculated by equation 3.18, JHRP 79-11.

Nomographs and/or charts for solution of the five dependent variables are given as Figures 3.31, 3.32, 3.33, 3.34 and 3.35, JHRP 79-11. Predictions seem compatible with those of Essigmann (JHRP 76-28) for density and unconfined undrained strength on a laboratory-compacted glacial silty clay. This is shown in Figures 3.35, 3.37 and 3.38, JHRP 79-11.

Lacking comparable data for a field-compacted highly plastic clay, this study provides the recommended model for determining stability of high embankments at the end of construction. The regression coefficients reported herein have limited application, but the functional forms of these equations should hold for many soils with plasticity similar to the test clay.

Task D - Compressibility of Field Compacted Clay

Background

This study investigates the compressibility behavior of a field compacted soil in the as-compacted and soaked conditions. The field compacted samples were incrementally loaded in the oedometer to determine the compactive prestress. Other field compacted samples were saturated by a back pressure technique in the oedometer under an equivalent embankment load and the 1-D volume changes produced by saturation were measured. A similar study on the compressibility characteristics of laboratory compacted samples from the same area soil had been conducted by DiBernardo (Task A), and the coupling of the relations for field compaction with those previously established for laboratory compaction is reported here.

Experimentation

The type of soil used was a plastic clay from shale, and field compaction was achieved using a Caterpillar Model 825 tamping roller and a RayGo Rascal Model 420C vibratory roller. Three energy levels and five molding water contents were used for each roller type to study their effects on compressibility behavior.

To examine the as-compacted compressibility characteristics, the compacted samples were trimmed to appropriate size and incrementally loaded in the oedometer. Compaction prestress values were interpreted from the e-log p curves; these values were

always less than the nominal roller pressures previously applied to the soil.

During the service life of an earthen embankment, environmental changes may effect an increase or decrease in the volume of the mass. In order to simulate the changes in the mass that may occur in service, other compacted samples were saturated under three levels of confinement, with the aid of vacuum and back pressure. The subsequent volume changes depended upon the compaction variables, as well as the confinement during saturation. These same samples were additionally loaded to define the soaked compressibilities.

The test procedures are described in detail in JHRP Report No. 81-14, and can be recreated by other interest parties through reference to JHRP 81-14.

Conclusions

With the construction of higher embankments becoming more common, it is increasingly more important to specify compaction procedures such that embankment settlement can be predicted and controlled for both the short and long term conditions. This study presents a model plan for field compacted clay and develops a rational method of predicting the field compaction response from laboratory tests.

The compression vs. time relations show that a large percentage of the as-compacted compression occurs within the first

minute of loading. This implies that fills settle under their own weight just about as rapidly as they are built and no post constructional compression within the fill due to self weight will occur. Settlement (or possibly heave) occurs when the fill becomes wetter in service. The amount of settlement or heave depends upon the compaction water content, compacted density(or void ratio) and the confinement. A prediction equation has been written for the St. Croix clay. The settlement (or heave) also depends upon the roller prestress, but the prediction equation is written in terms of original compaction variables for convenience. The compactive prestress value is very important to shear strength, and it has been found to vary with compaction water content and roller pressure.

The most important finding is that compressibility for both laboratory compacted samples and field-compacted samples depends on the same variables. Only the regression constants are different. The similarity of laboratory and field compacted relationships allows the predictions to be simply related in plots (Figures 32, 33 and 34, JHRP 81-14). If the laboratory samples are compacted in a realistic mode (we recommend kneading) field compacted responses can be predicted from them. Finally, behavior for soils similar to the St. Croix clay can be predicted by comparison of compaction curves and plasticity. A statistical prediction procedure was developed which is illustrated in Figure 35 and Figure 36, JHRP 81-14.

Task E - Strength of Field Compacted Clayey Embankments

Background

The purpose of this study was to investigate the relationships among the compaction variables and the shearing behavior of a field compacted St. Croix clay from shale. The strength tests were performed by unconsolidated-undrained (UU) and saturated consolidated-undrained ($\overline{\text{CIU}}$) triaxials. These were run at various confining pressures to approximate the end of construction and long term conditions at several embankment depths. Statistical prediction models were produced for dry density ($\hat{\rho}_d$), as-compacted strength ($\hat{q}_c$), percent volume change due to saturation and consolidation ($\Delta\hat{V}/V_o$, %), Skempton's A parameter at failure ($\hat{A}_f$) and the effective stress strength parameters (c' and ϕ'), in terms of the field compaction variables. Relations for field compaction were coupled with those previously defined for laboratory compaction of a St. Croix area clay.

Experimentation

Samples for the triaxial tests were taken from the ten test pads, which had been compacted to three levels of effort, at five levels of water content, and by two kinds of rollers. The soil used in the field test pads was a plastic residual clay from shale, and field compaction was achieved using a Caterpillar Model 825 tamping roller and a RayGo Rascal Model 420C vibratory roller.

Laboratory tests were run for as-compacted strength, volume change due to consolidation and saturation, and saturated undrained strength on the sample procured from the test pads. The experimental test parameters were related to the compaction variables, and statistically valid prediction equations were produced.

The test procedures are described in detail in JHRP Report No. 82-1 and can be repeated in detail by reference to this report.

Conclusions

The as-compacted strength of the field compacted soil is a function of initial void ratio, confining pressure and water content. The volume changes occurring during shear for as-compacted samples are significant, especially for samples of low water content and low dry density. The volume changes due to saturation are found to be functions of initial void ratio, water content and estimated compactive prestress. Lower confining pressures and higher estimated compactive prestress for a given water content and dry density produce a greater swell potential for the field compacted St. Croix clay. Skempton's A parameter at failure is a function of initial void ratio and OCR (P_s/σ'_c). The predicted effective stress strength parameters, c' and ϕ', are functions of compactive water content and initial void ratio.

The statistically valid prediction models for the field compacted St. Croix clay are summarized in Table 3.22, JHRP 82-1. for this field compacted clay.

Two methods were developed for the prediction of field compacted relationships for one soil from laboratory tests on a somewhat different soil. The first step is to translate the laboratory compacted relations to those appropriate for the soil involved in the field compaction. The translation factor is the ratio of Standard Proctor optimum moisture content values. Applying the translation factor allows the selection of appropriate values for the statistical development in the equations 3-24, 3-25, and 3-27 (JHRP 82-1).

Task EE - Soil Compaction Specification Procedure for Desired Field Strength Response

Background

This study intended to evaluate the variables controlling the behavior of the field compacted clay of low plasticity (A-4) previously studied in the laboratory in Phase I. A concentrated effort was to be made to obtain the necessary field data to allow the useful correlation with laboratory data, as heretofore not possible. This, then, was intended to lead to a prediction of in-service field behavior, as well as a technique for preparing specifications to assure a desired field behavior parameter.

Experimentation

A field test pad was constructed under somewhat controlled conditions using both a sheepsfoot roller (Case, Model 815) and a rubber-tired roller (Ferguson, Model RT-2511). Samples were taken and tested for as-compacted and soaked unconfined compression. It was found that the control of construction secured only wet-of-optimum data, and those were analyzed statistically and compared with the laboratory data from Phase I investigations. Many techniques were attempted for the prediction and specification design procedures.

Results and Conclusions

Once again, water content was found to be important in the magnitude of the achieved dry density and strength. Variability

in strength was large but predictable from the magnitude and variability of the compaction variables, especially the range of actual water content of the lift. A computer program was developed for the variability calculation.

A procedure was developed to account for both the desired strength (as-compacted or soaked) and its variability for development of the specification for compaction. This technique indicates how much compaction to specify to obtain the desired strength, with assurance.

A computer tabulation format was developed to list independent compaction variables and their uncertainty, the expected strength response, and the possible range of the expected strength. From such a format, quality assurance engineers can predict the performance of the compacted soil using inspection test results.

A User's Guide was developed for the application of these results.

Tasks FF-GG - Prediction and Control of Field Swell Pressures of Compacted Medium Plastic Clay

Background

Compaction is an effective means of improving the behavior properties of clayey soils in the field. State-of-the-art procedures do not specify the actual behavior property parameters to be achieved; they have been obtained only by inference by the engineer, often from only laboratory data. The changes in behavior that occur with changes in environmental conditions over the life of the structure are also mostly included in the engineer's consideration by inference. In the case of compacted clay, there is induced in the soil a tendency to swell; the soil may swell in volume when water becomes available and this may negate many of the positive benefits of compaction. There is, indeed, the need to better predict and control field behavior.

Experimentation

Field data were obtained from samples from a test pad constructed of a residual clay from shale (the soil used in Tasks A, B, C, D, E). Two rollers were used: a) Caterpillar Model 825 segmented pad roller; b) RayGo Rascal Model 420C vibratory segmented pad roller. The same soil was also tested in the laboratory using both kneading (Hveem-type) compactor and impact (Proctor-type) procedures.

Constant-volume swell tests were performed in benchtop consolidometers, and the effects of compaction variables were studied systematically. The data were evaluated statistically, and the laboratory vs. field correlation was sought. Many techniques were attempted for the prediction and specification design procedure.

Results and Conclusions

Swell pressure magnitudes and variability were found related to water content, compaction energy, dry density, and interactions among the variables. Laboratory and field relationships were quite different; laboratory relations were similar among themselves. Field samples exhibited larger variabilities. Variabilities were large but predictable.

Prediction models were prepared for field swell pressure. As presented in chart form, they enable the engineer to select compaction variables that will control the field swell pressure to a predetermined maximum value.

A computer tabulation format was also created to list the magnitudes of the compaction variables and their uncertainty, the expected mean swell pressure, and the variability in swell pressure. Quality assurance engineers can use this format to predict the performance of the compacted clay from inspection test results. A method has also been proposed to apply adjustments to

these results for soils that have somewhat different compaction characteristics.

A User's Guide was developed for the application of the results.

Task FFF - Design of Compacted Clay Embankments for Improved Stability and Settlement Performance

Background

Purdue University has a long standing interest in the development of user-oriented slope stability computer programs. One of the first developments was the SLOPE program package (Carter, 1971) consisting of four separate programs. The subsequent development was the STABL program (Siegel, 1975 and Boutrup, 1977). This program can evaluate the factor of safety of slopes of almost any description and shape. STABL is used on a regular basis by the Indiana Department of Highways for routine evaluation of slope stability. The purpose of this task was two fold. First, to illustrate how the results of the compacted clay investigation can be best used in the design and stability analysis of compacted clay embankments. Second, to complete the analysis package by supplying computer programs for the calculation of embankment settlement.

Results and Conclusions

The alternatives of specifying compaction procedures or compaction results were compared and an hybrid approach of specifying compaction that integrates the advantages of these two approaches was introduced. The optimum moisture content varies along the line of optimums, and as the compactive effort increases, the optimum moisture content decreases. When a compactor imparts a compactive effort associated with an optimum

moisture content equal to the water content in the field, the compaction process is efficient. In the hybrid approach of compaction specification, the compactive effort is specified so that the corresponding optimum moisture content is equal to the expected compaction water content.

Embankment side slope designs were illustrated for short and long term conditions using laboratory compacted shear strength data. In these examples the embankment material was assumed to be compacted St. Croix clay and the strength parameters for short and long term conditions were obtained from the reports by Weitzel and Lovell (1979) and Johnson and Lovell (1979), respectively. During the course of this project several improvements were made to the STABL program:

- The Simplified Bishop factor of safety option was recoded to correct various difficulties discovered by STABL users.
- Recommendations were made to avoid common errors in the use of STABL (surfaces with unacceptable shapes, direction limits on benched slopes, direction of surface generation, etc.).
- A methodology was developed to adjust the Simplified Janbu factor of safety (used in several STABL options) to more familiar definitions of the factor of safety.

Geometric and probabilistic interpretation of the factor of safety was introduced to demonstrate their usefulness as alternatives and/or supplements to the conventional strength factor of safety. For a given height (slope) of an embankment the geometric factor of safety can be defined as the ratio if the slope (height) at limit equilibrium to the actual slope (height) of the embankment. The probabilistic approach takes into account the variability of material parameters and provide the reliability of the slope corresponding to the computed factor of safety.

Computer programs have been developed to compute the magnitude of settlement that occur within the embankment itself as well as of the consolidation settlement of fine grained soil layers below the embankment. Displacements within the fill are caused principally by compression under the fill body force and volume change due to increase in moisture content of the compacted embankment. Programs were also developed to estimate the time-rate of consolidation settlement of the compressible soil layers beneath an embankment. These programs, in conjunction with STABL, form an analysis package for the design of embankments. User's manuals and listings were provided for all these computer programs.

Task HH - In-Service Sampling and Record Testing of Existing Embankments

Background

The other tasks of this project are concerned with forecasts of behavior of the field compacted soil. These predictions cover not only the as-compacted state, but they also include the behavior after soaking has occurred in-service. This task has the objective of validating the prediction techniques of the other tasks by suitable sampling and testing.

Investigation

Several existing embankments were identified with the assistance of IDOH personnel, as being composed of soil similar to those of this project and having been compacted with similar rollers as well. Construction records of IDOH were searched to locate the results of inspection testing and similar such details; incompleteness of data was one reason to discard possible projects. Where adequate data were present, the embankments were sampled using personnel and equipment of IDOH; this sampling was scheduled to fit into the regular IDOH work program on an "as-time-permits" basis. The data were inserted into the prediction techniques of this project; forecasts were made for the behavior parameters to be expected for the soils at the locations of the sampling. The samples were tested in the laboratory to

obtain the actual parameters. Comparisons were made between predicted and actual magnitudes of parameters.

Conclusions

Inspection testing on construction projects in the past has been pointed to locating improperly compacted portions of embankments. This project's focus is on the range of characteristics of the embankment, not a bias towards the poorly compacted. This is not a criticism in any way; it is a recognition that purposes are different. As a consequence, a number of prospective locations were discarded because of too few available data.

A separate section of this report contains a tabulation of the results of the comparisons made (see Table 4). It is believed the comparisons are favorable and point to the potential quality and usefulness of the techniques recommended by this project.

Applications of the Results

Introduction

The major factor that appears to face the engineer when considering field behavior is the large variability that appears to be present in the field compacted soil. The data from this project continually showed large variability. It cannot be ignored. Application of the results of this study are intended to be of 2 kinds: 1) design engineering - to allow the engineer to create a specification for field compaction that will assure him of a specific desired upper, or lower, bound on specific parameter(s) magnitudes; 2) quality assurance - to allow a prediction of the magnitude(s) of a specific parameter(s) knowing field inspection results. The former is expected where the borrow is known in advance of construction. The latter is expected where the borrow is not well known prior to construction. The results may be used in two ways: (a) statistical interpretation and (b) general or judgmental.

These applications require the data base of this study. Uses will apply only to the soils and equipment from which the data base was prepared, although it is believed some generalization is possible, as is suggested by the procedures herein. Successful use requires that an understanding be reached of the role of variability of the parameter as well as its identifiable sources.

This study examined the data from the test pads and established variabilities from them. The next section will show how the calculations were made as well as charts which portray the variability magnitudes. The data from the charts were then manipulated to develop the charts that are presented in the section on design engineering applications. In addition the research provides valuable guidance in the areas of laboratory testing, interpretation of test data, and in the development of statistical relationships. These data are principally available in the individual task reports.

Variability in Parameters

The construction operation created variability in test pad water content, and the range of that water content was measured. The average range was considered the expected range that must be accounted for in prediction. The number of passes of equipment was counted; thus, there is no variability in this parameter. The dry density variability to be expected depended on the confidence limits one wished to place upon that parameter. A 95 percent level was used in this study. Dry density was a function of water content and compaction energy. For each compaction energy level the dry density variability was calculated at 2 points: (1) w + half-range; (2) w - half-range. The largest variability of the 2 values calculated was taken as the expected variability. With the dry density variability determined, the variability was calculated for the parameter desired (e.g. as-compacted strength). The evaluation of the parameter thus includes

variabilities in the compaction variables that "created" the soil. Thus, the following 4 conditions need to be considered:

E	w	γ_D	V(parameter)
E	w + (half range)	$\hat{\gamma}_D + V(\hat{\gamma}_D)$	$V(\text{parameter})_1$
E	w + (half range)	$\hat{\gamma}_D - V(\hat{\gamma}_D)$	$V(\text{parameter})_2$
E	w - (half range)	$\hat{\gamma}_D + V(\hat{\gamma}_D)$	$V(\text{parameter})_3$
E	w - (half range)	$\hat{\gamma}_D - V(\hat{\gamma}_D)$	$V(\text{parameter})_4$

where E represents the energy level being considered; [w + (half range)] represents maximum water content expected; [$\hat{\gamma}_D + V(\hat{\gamma}_D)$] represents the maximum predicted dry density; and V (parameter) represents one predicted magnitude of the parameter variability, which prediction accounts for 2 possibilities of compaction variables. The largest value of the 4 predictions for parameter variability was considered as the variability to be expected for the response level being calculated. Again, a confidence level of 0.95 was used in this calculation of variability.

Using the statistical models prepared for variability for each parameter, the computer program presented in Appendix A was used for calculations. The program explains the method of calculation for one particular parameter. For different parameters the program may be modified for their calculation with different input data. The program uses the following parameters:

$$V(\text{parâmeter}) = (\lambda)(s)\sqrt{X'_p\,[X^1 X]^{-1}\,X_p}$$

where V (parameter) = variability in the parameter as defined for 1 set of water content, dry density, and energy;

X_p = a column vector containing the values of the independent variables for which the calculation is sought;

X'_p = X_p transposed

s = the square root of the mean square error from the regression model for the parameter data set-obtained from computer program REGRESSION

λ = the appropriate t-statistic dependent upon the number of samples of the data set and the confidence level desired by the engineer

$$[X'X]^{-1} = \begin{vmatrix} \frac{1}{n} + \frac{1}{S^2} R'VR & -\frac{1}{S^2} R'V \\ -\frac{1}{S^2} VR & \frac{1}{S^2} V \end{vmatrix}$$

where n = number of samples of the data set

s = as defined earlier, above

R = a column vector of the means of the independent variables used in the

regression model - obtained from

output of computer program REGRESSION

R' = R transposed

V = the symmetric variance/covariance matrix

of the regression model

The variabilities in parameters were calculated and charts were developed. These charts are presented as Figures 1 through 20; they may be used directly. If the variables of a given situation differ from those on the figures, the program of Appendix A must be used to obtain the data from which charts may also be prepared.

Alternately, rather than measuring or specifying the engineer may simply wish to estimate the variability in any input (independent) variable, depending upon the degrees of control which he believes will operate for site specific compaction variables. Insertion into the equations or charts developed in this research will create the prediction of the parameter variability.

Design Engineering

Prediction equations have been prepared for the parameters developed for the soils and equipment of this study. These equations for magnitude of parameter are tabulated in Appendix B.

The charts for variability of each parameter were combined with the equations for magnitude to develop charts for use by the design engineer. Substitutions of many combinations of variables

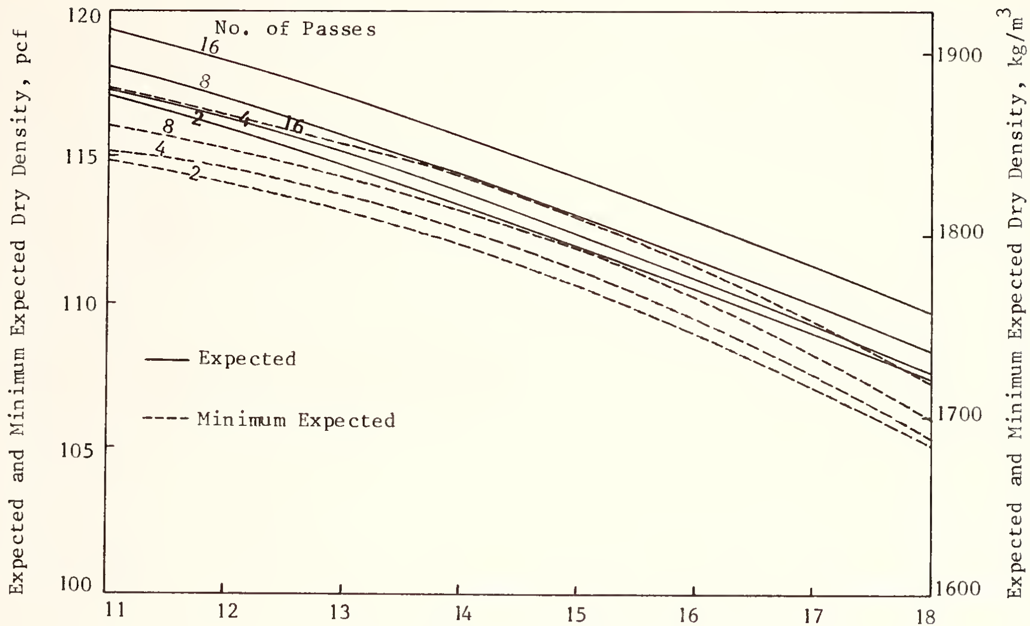

Figure 1 Expected and Minimum Expected Dry Density: Rubber Tired Roller, As Compacted; (A-4)

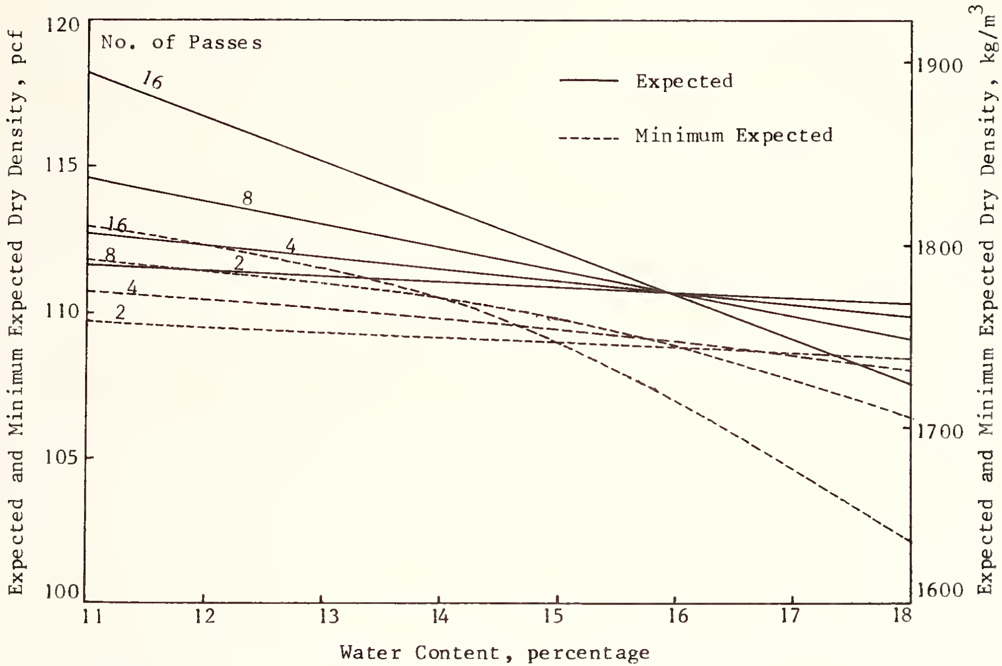

Figure 2 Expected and Minimum Expected Dry Density: Rubber Tired Roller; Soaked (A-4)

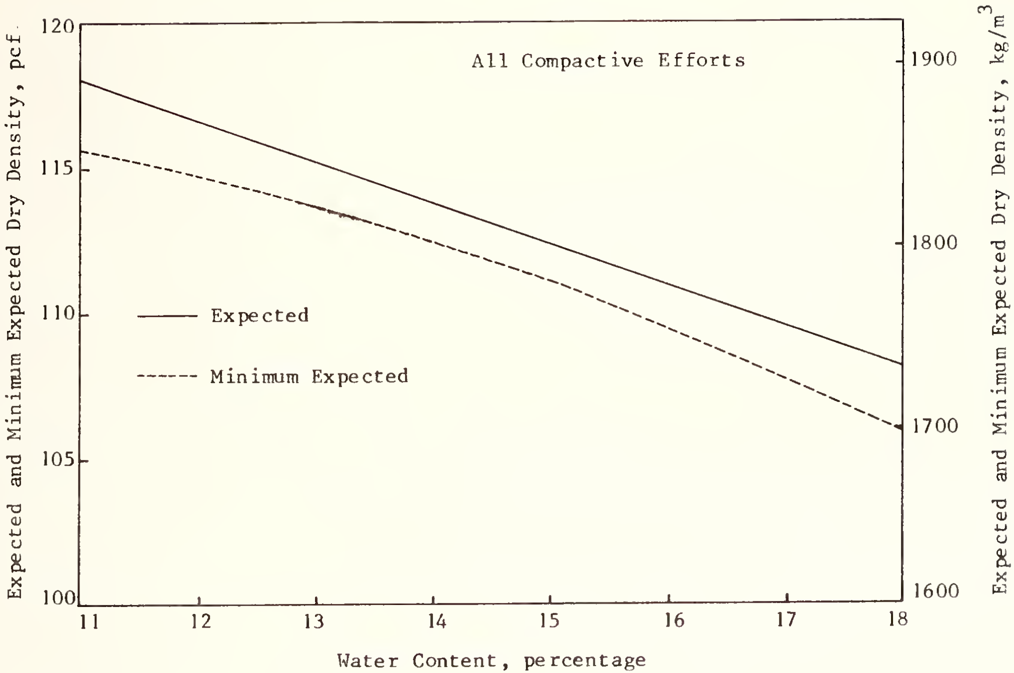

Figure 3 Expected and Minimum Expected Field Dry Density: Sheepsfoot Roller, As Compacted; (A-4)

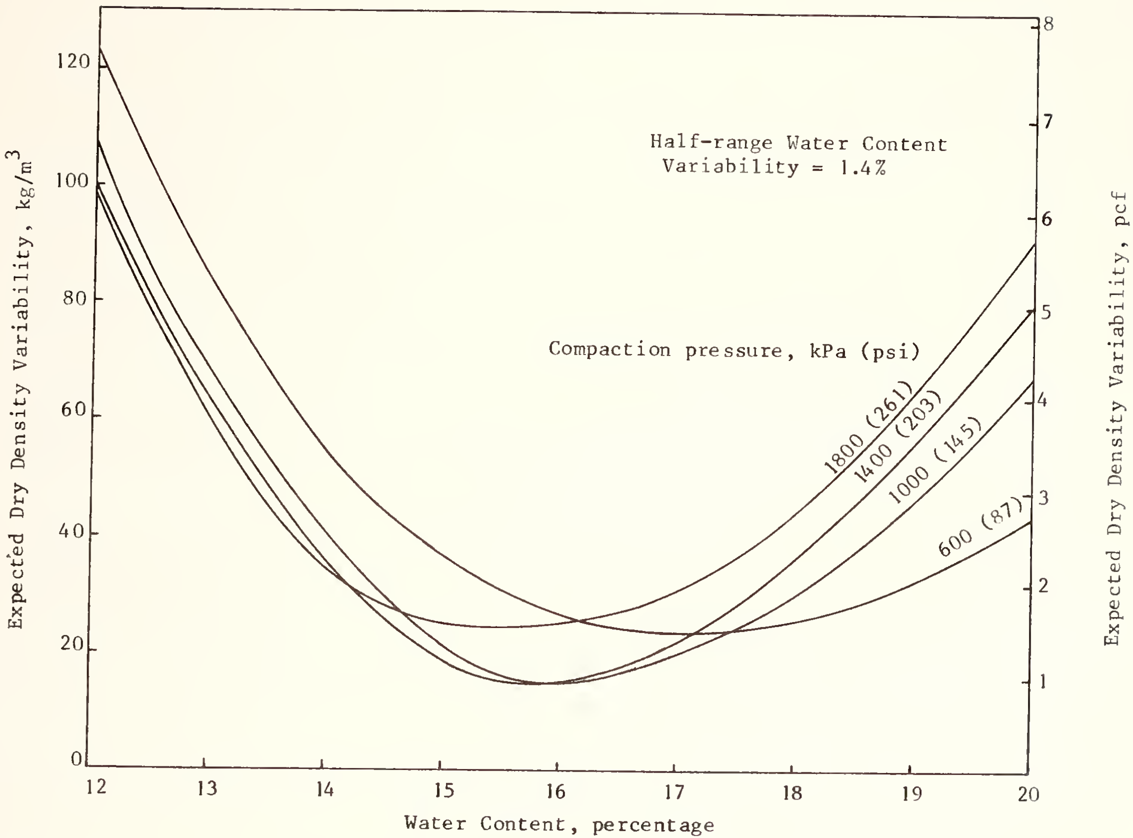

Figure 4 Expected Field Dry Density Variability

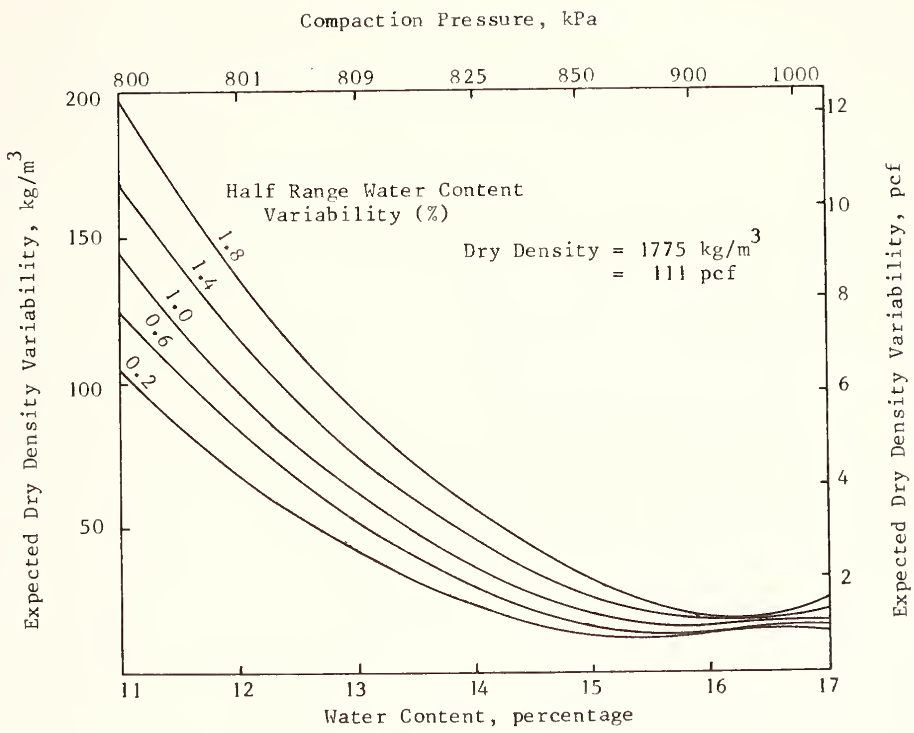

Figure 5 Expected Field Dry Density Variability; ρ_d = 1775 kg/m^3 (111 pcf); (A-6, A-7-6)

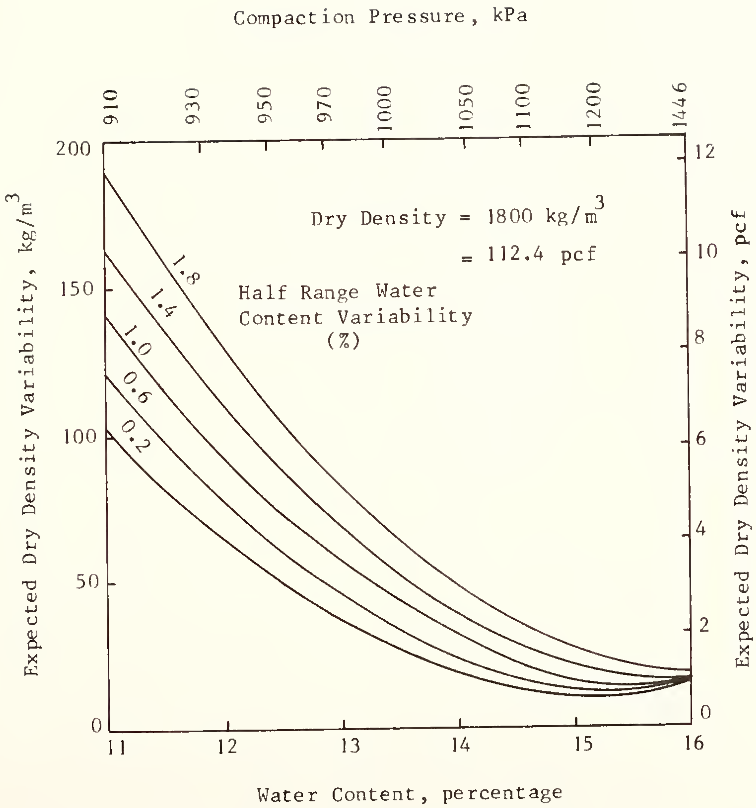

Figure 6 Expected Field Dry Density Variability; ρ_d = 1800 kg/m^3 (112.4 pcf); (A-6, A-7-6)

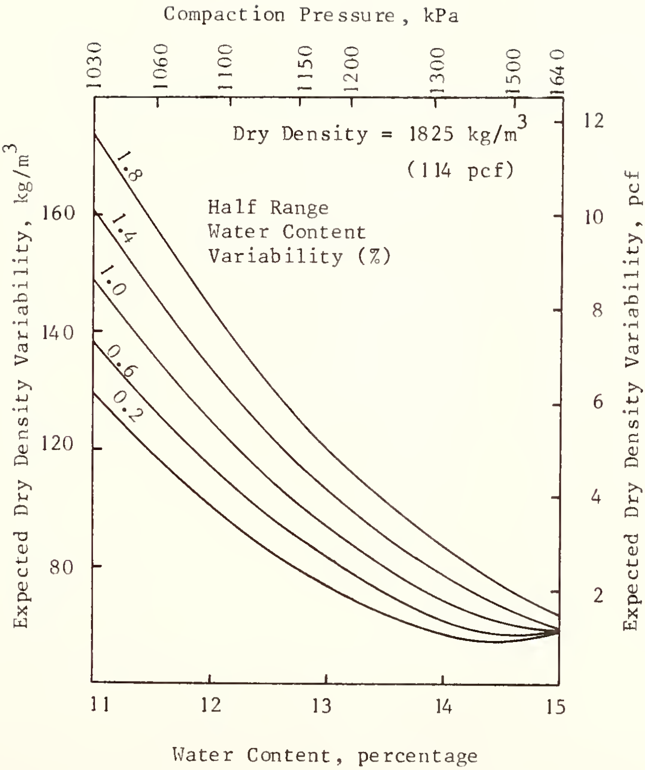

Figure 7 Expected Field Dry Density Variability; ρ_a = 1825 kg/m^3 (114 pcf); (A-6, A-7-6)

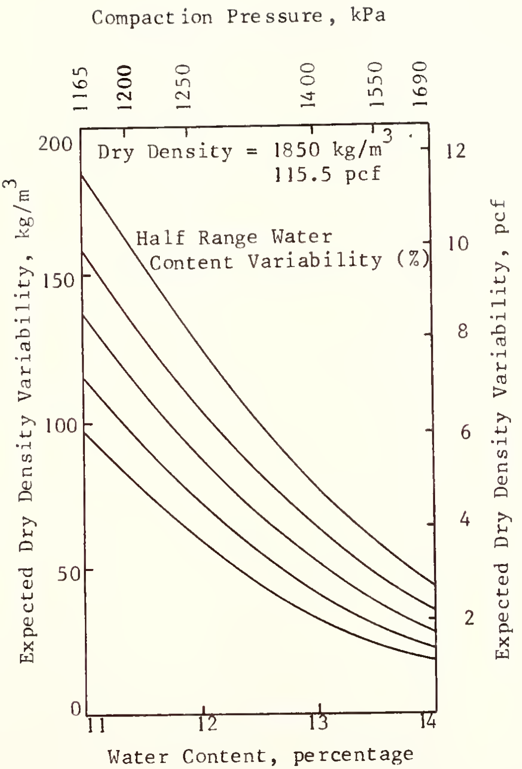

Figure 8 Expected Field Dry Density Variability
ρ_d = 1850 kg/m^3 (115.5 pcf); (A-6, A-7-6)

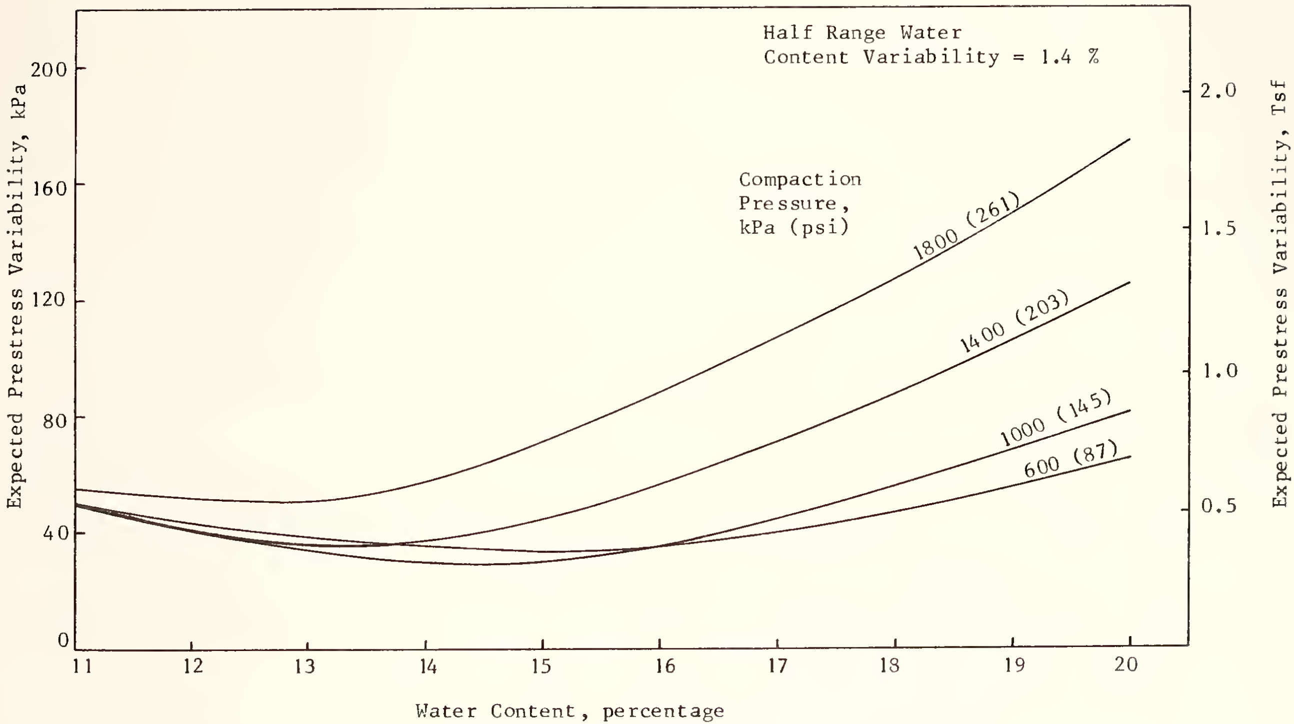

Figure 9 Expected Field Prestress Variability; (A-6, A-7-6)

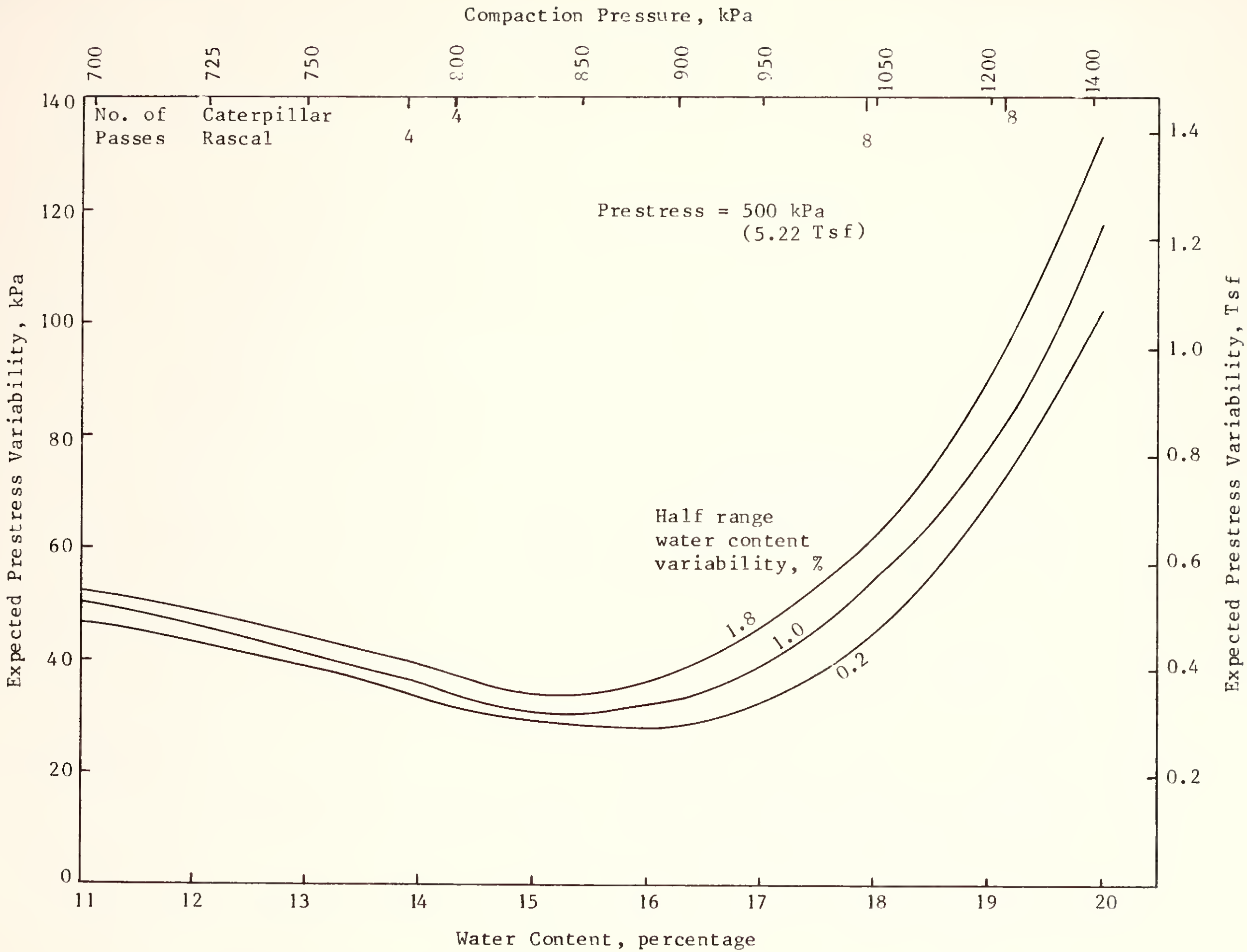

Figure 10 Expected Field Prestress Variability; Ps = 500 kPa (5.22 tsf); (A-6, A-7-6)

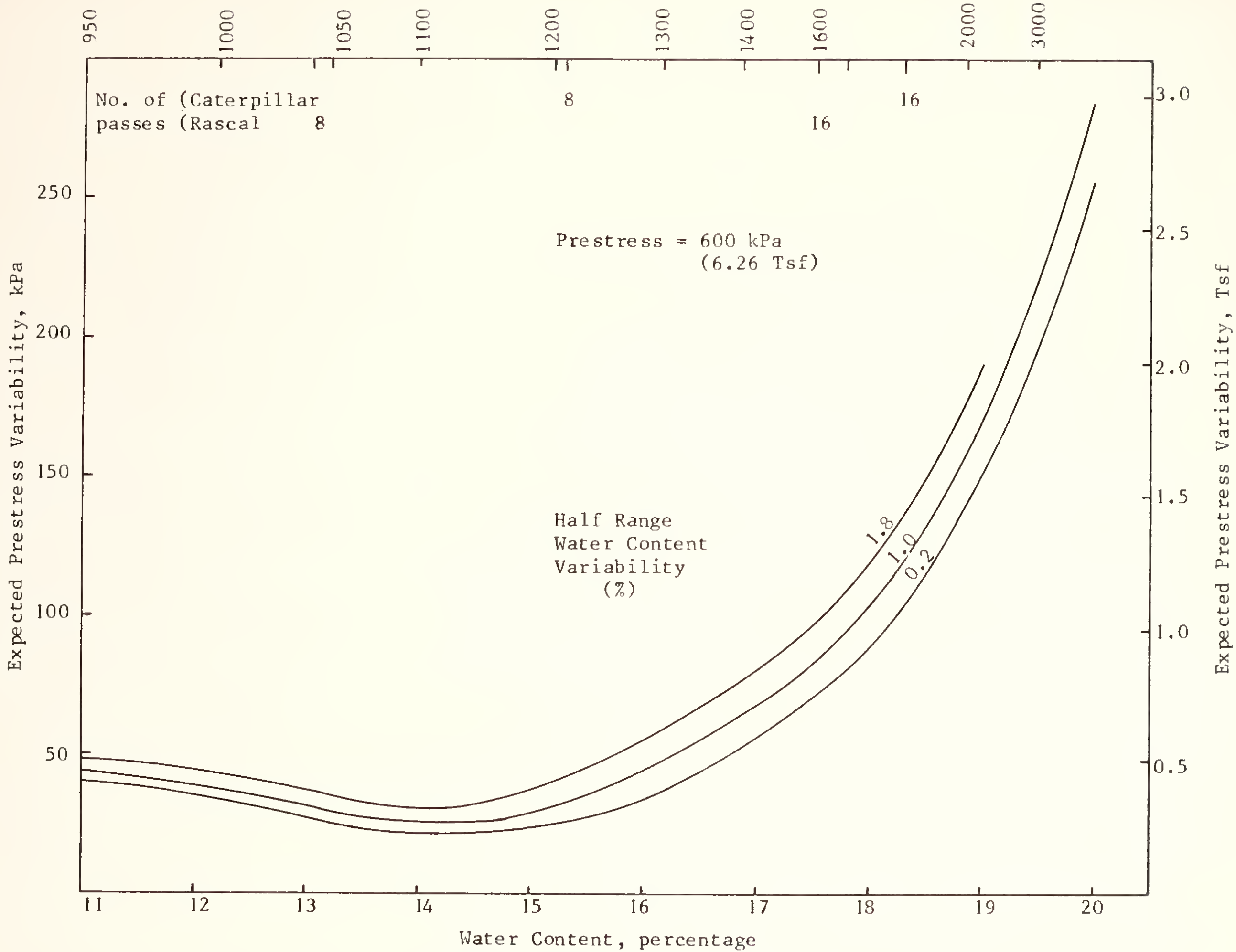

Figure 11 Expected Field Prestress Variability; P_s = 600 kPa (626 Tsf); (A-6, A-7-6)

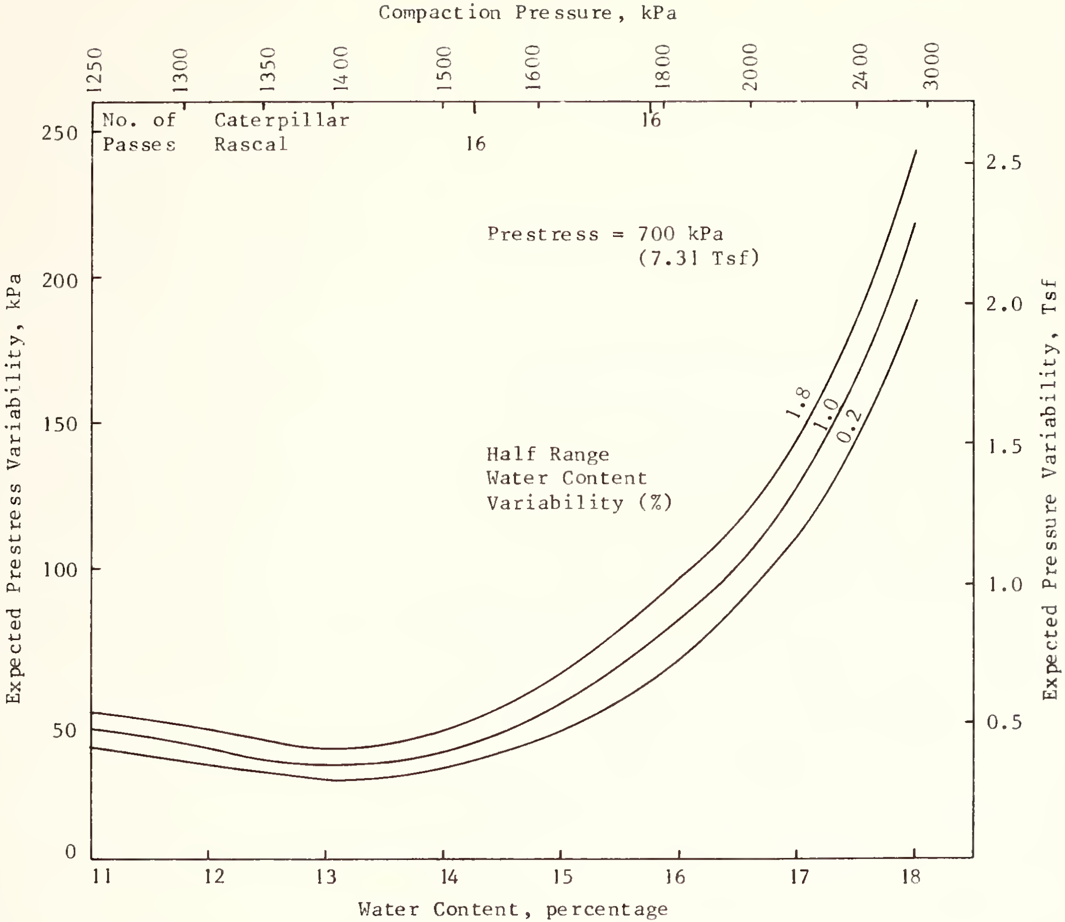

Figure 12 Expected Field Prestress Variability; P_s = 700 kPa (7.31 Tsf); (A-6, A-7-6)

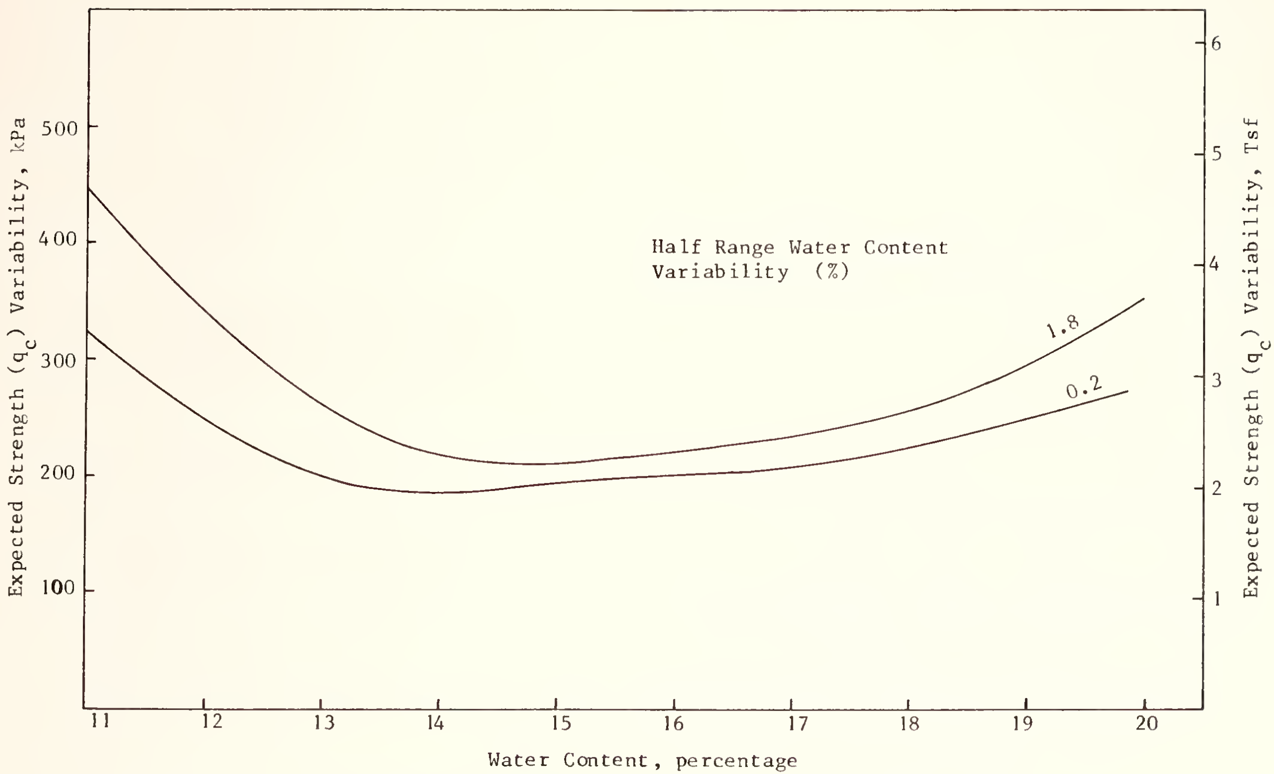

Figure 13 Expected Field Strength Variability (A-6, A-7-6)

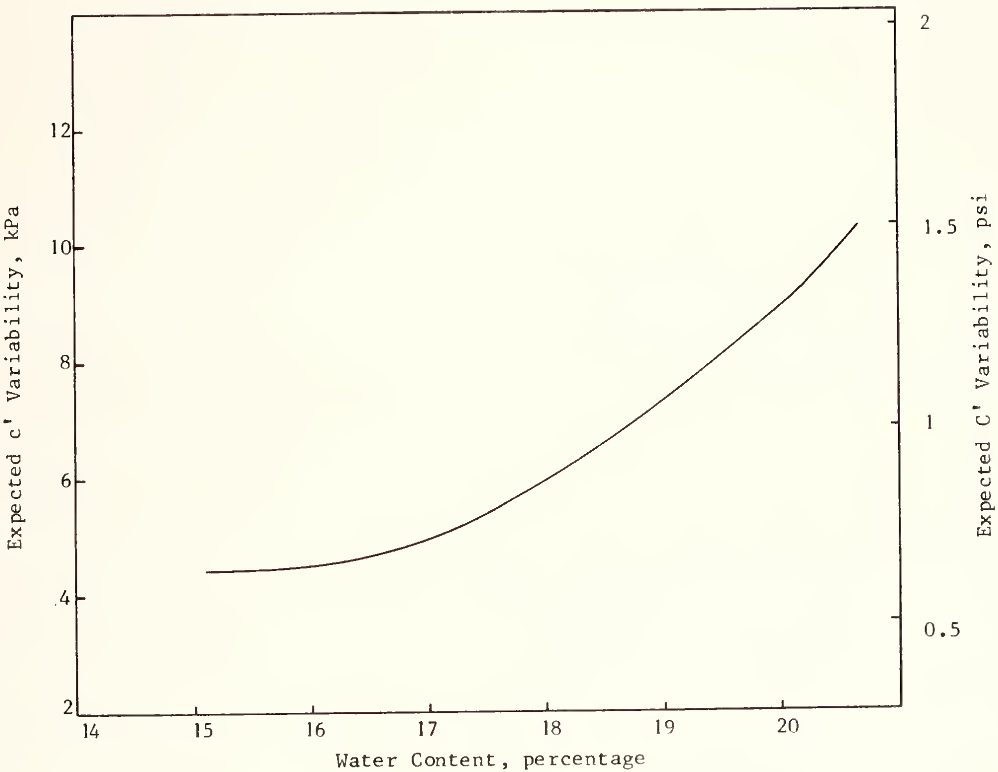

Figure 14 Expected Field c' Variability

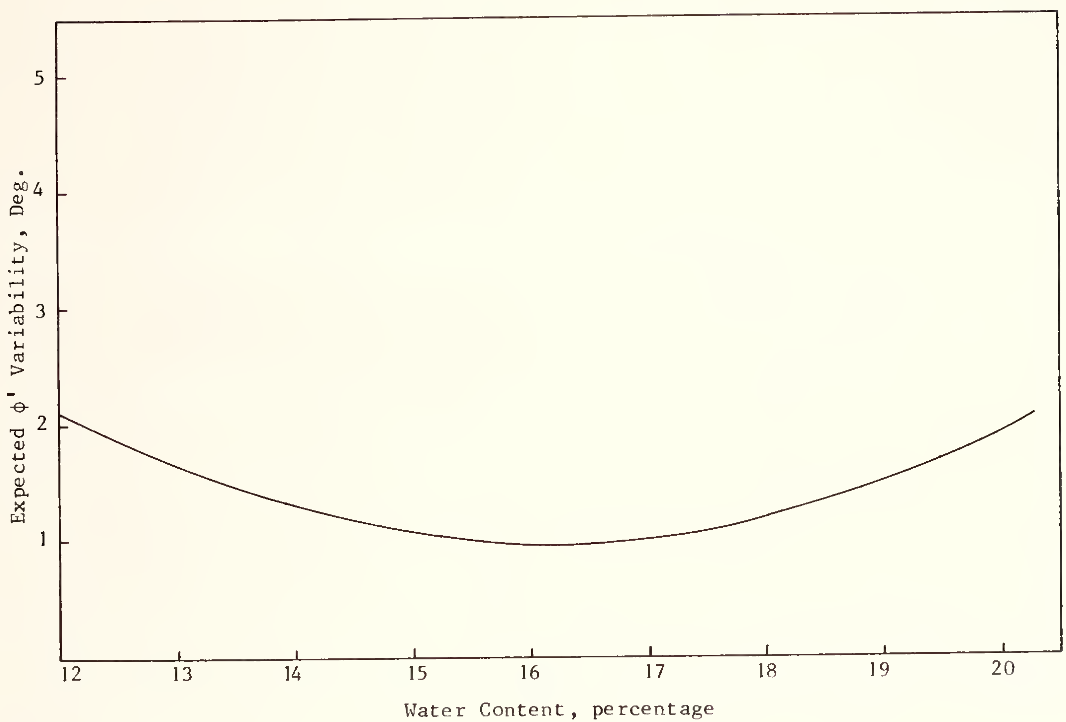

Figure 15 Expected Field ϕ' Variability

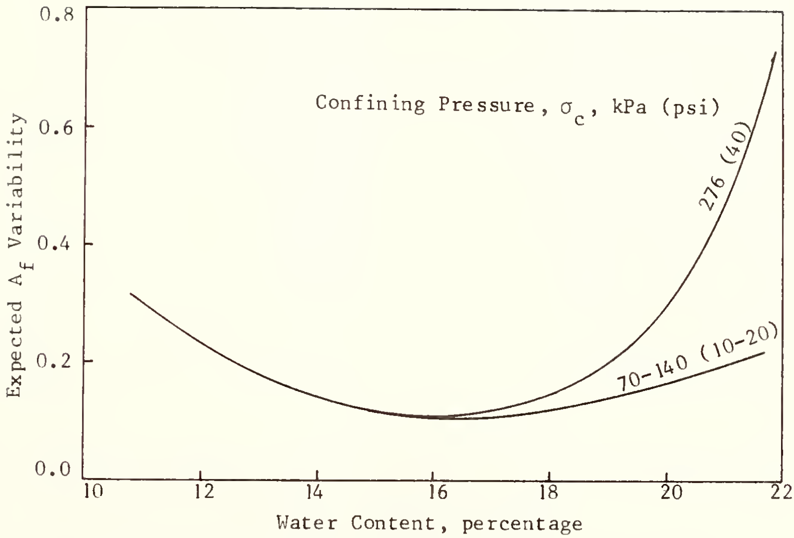

Figure 16 Expected Field A_f Variability, (A-6, A-7-6)

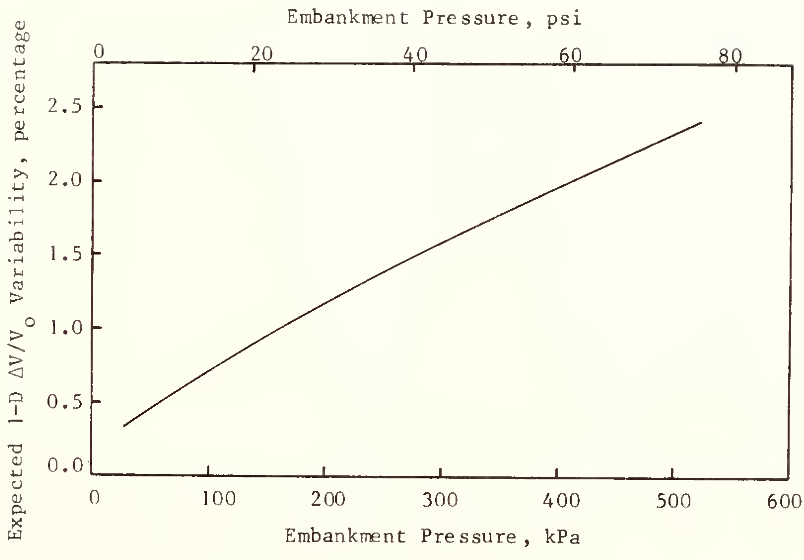

Figure 17 Expected Field (1-D) $\frac{\Delta V}{V_o}$ Variability; (A-6, A-7-6)

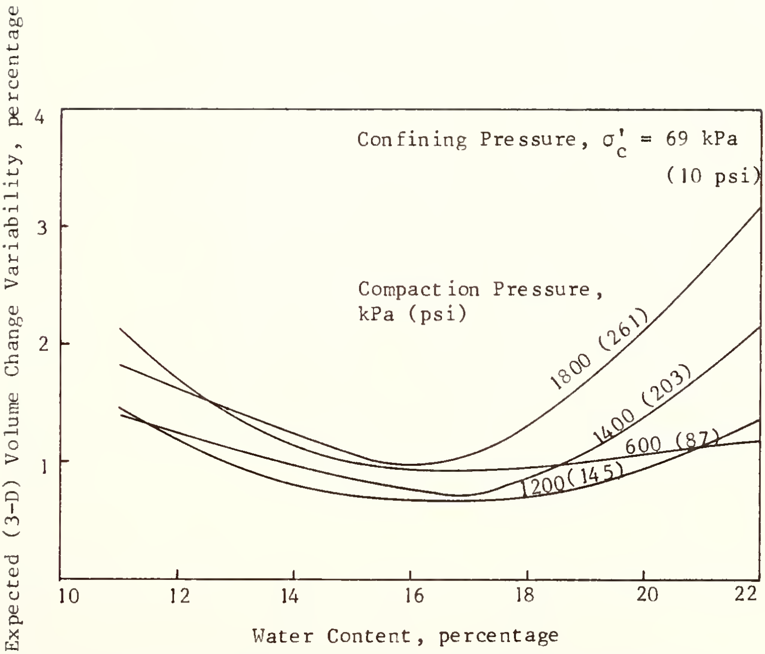

Figure 18 Expected Field (3-D) Volume Change Variability; σ_c' = 69 kPa (10 psi); (A-6, A-7-6)

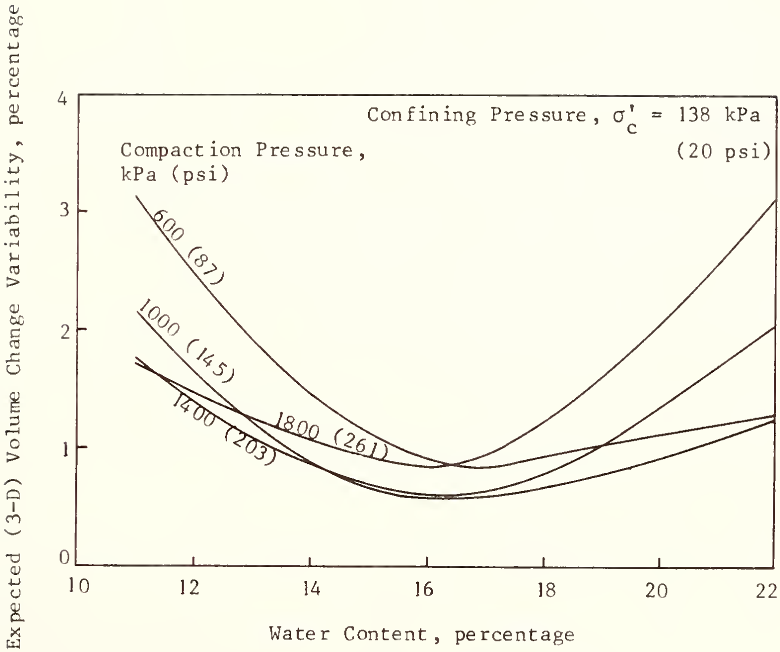

Figure 19 Expected Field (3-D) Volume Change Variability; $\sigma_c' = 138$ kPa (20 psi); (A-6, A-7-6)

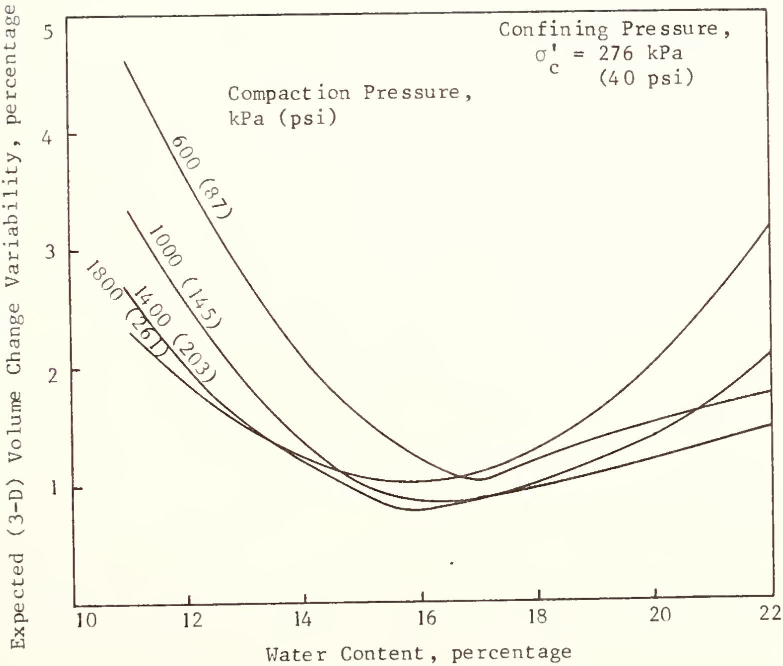

Figure 20 Expected Field (3-D) Volume Change Variability; σ'_c = 276 kPa (40 psi); (A-6, A-7-6)

allowed the creation of charts that show the expected (mean) value of the parameter and the minimum (or maximum) value; i.e., the mean value ± the variability. These charts are presented in Figures 21 through 37 as prepared for this study.

The charts may contain reference to number of compactor passes or to the compaction pressure applied to the soil. The two are related to each other. To allow conversion of compaction pressure to number of passes of the field equipment of this study Table 1 is presented.

The design chart is easily used. The engineer selects a desired magnitude of parameter that he can be assured will be present; this represents a lowest magnitude of strength, for example, or a maximum magnitude of swell pressure. Knowing, or fixing by specification, the range of water content present in the as-placed lift as well as the roller to be used, the chart tells him the water content at which the soil is to be compacted and the number of passes of the equipment that is required. These form the basis for the specification. The input to the chart is the property the engineer desires for the field soil; that is the major contribution of this work, to allow the engineer to, in essence, specify his property parameter. He can determine the combination of compaction variables to yield that parameter as well as determine the costs for using different magnitudes of property parameter(s) for his project. Indeed, now the engineer can select his suite of desired properties. Each will suggest different compaction variables. The engineer can

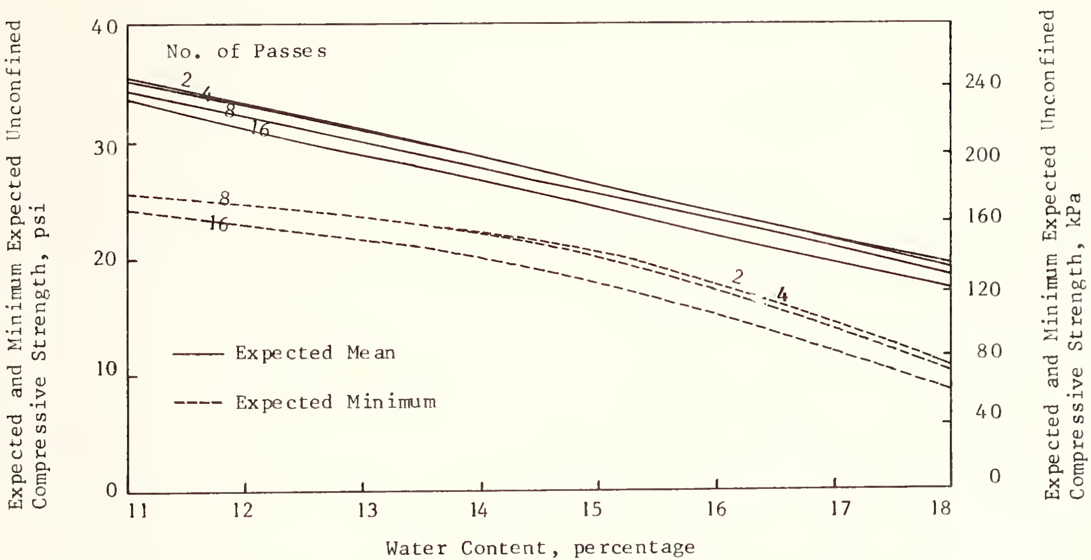

Figure 21 Design Chart for Unconfined Compressive Strength; Rubber Tired Roller; As Compacted; (A-4)

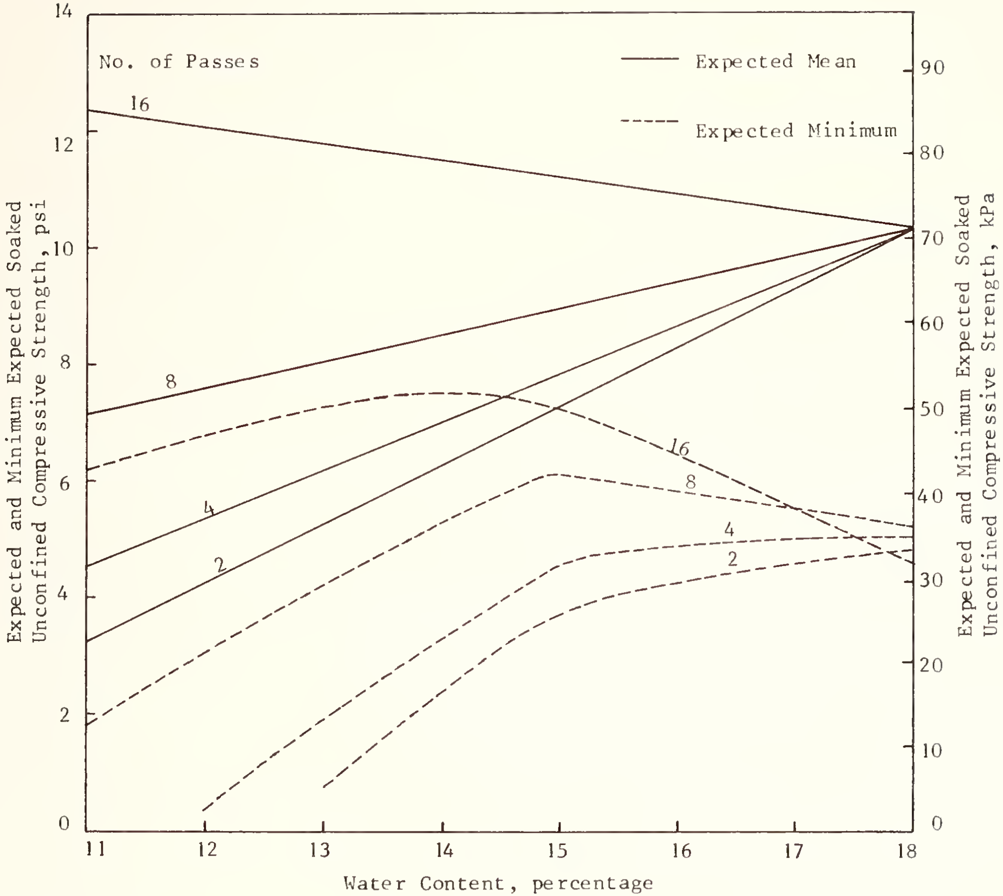

Figure 22 Design Chart for Unconfined Compressive Strength; Rubber Tired Roller; Soaked; (A-4)

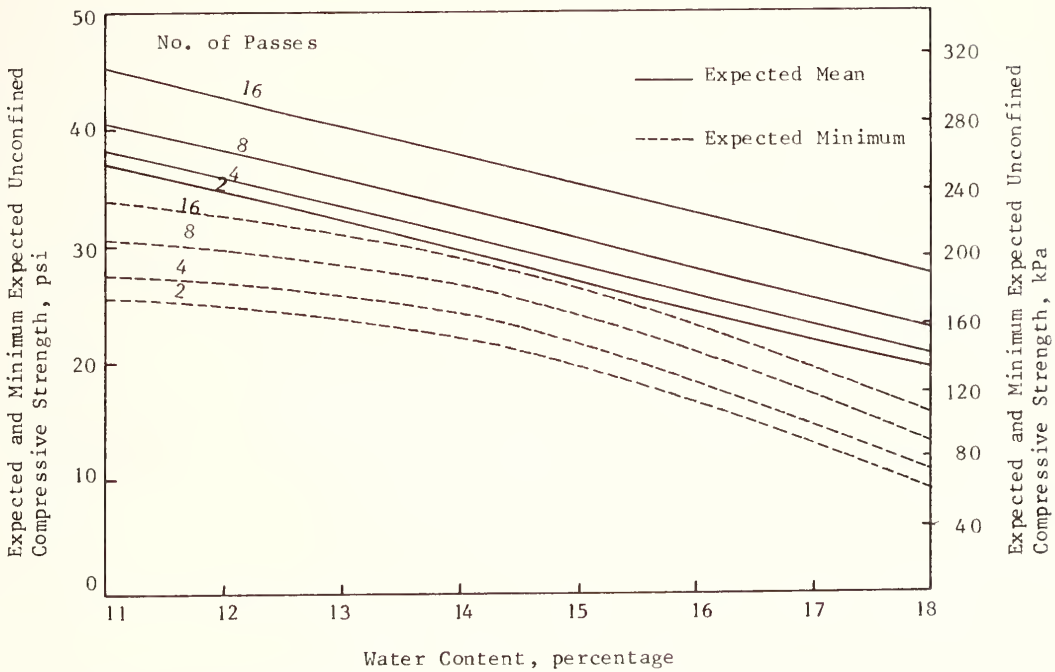

Figure 23 Design Chart for Unconfined Compressive Strength; Sheepsfoot Roller, As Compacted; (A-4)

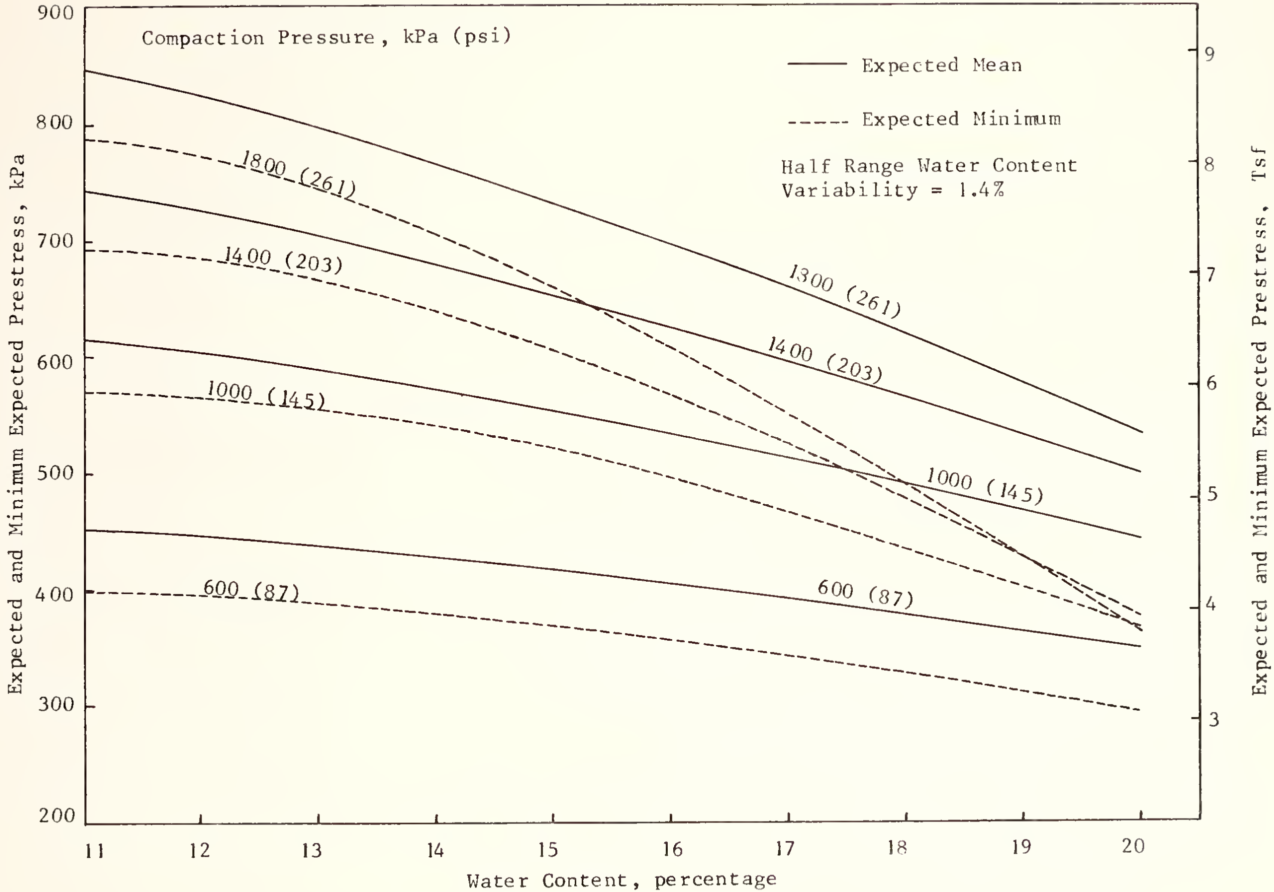

Figure 24 Design Chart for Field Prestress;
(A-6, A-7-6)

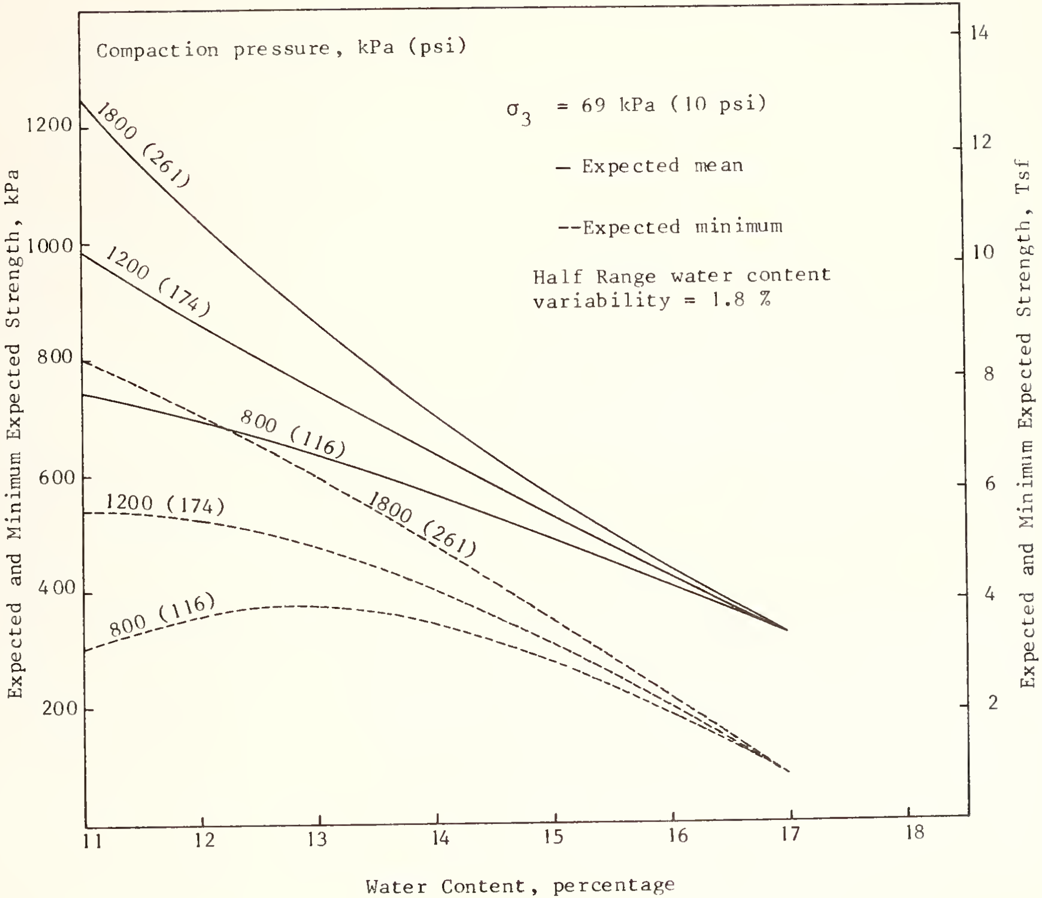

Figure 25 Design Chart for Field Confined Undrained Strength; σ_3 = 69 kPa (10 psi); (A-6, A-7-6)

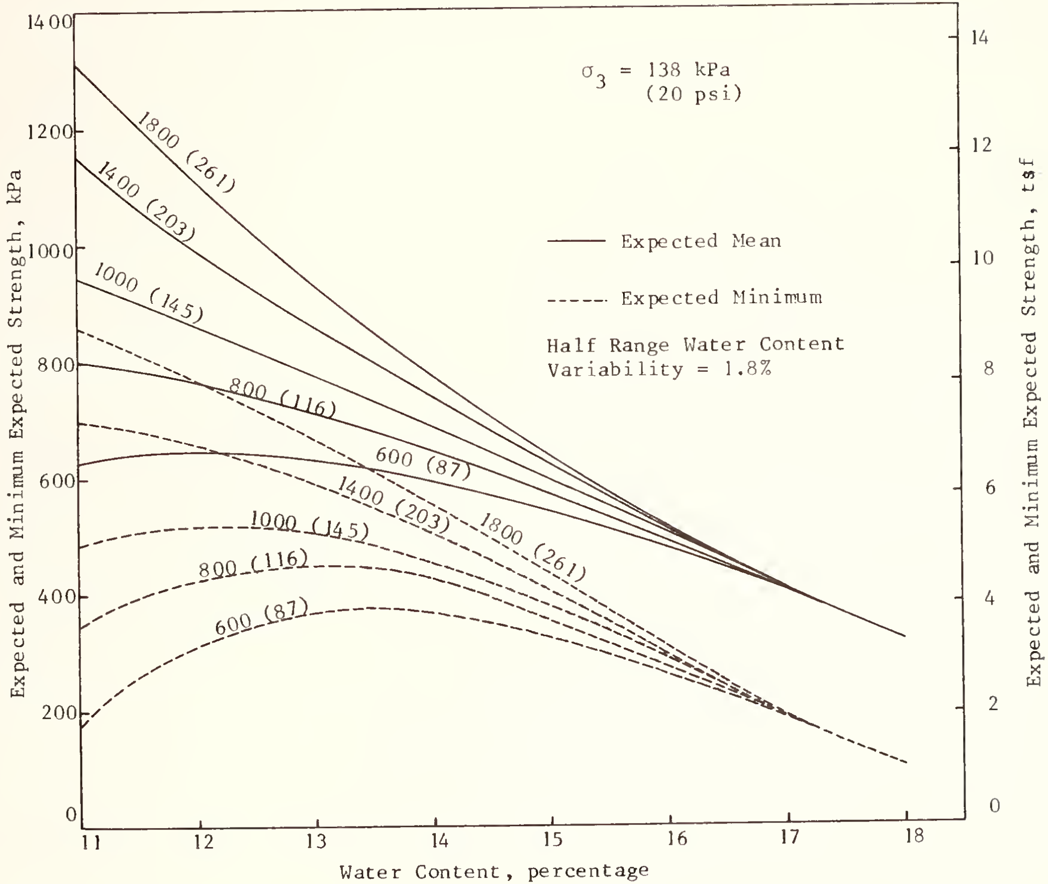

Figure 26 Design Chart for Field Confined Undrained Strength; σ_3 = 138 kPa (20 psi); (A-6, A-7-6)

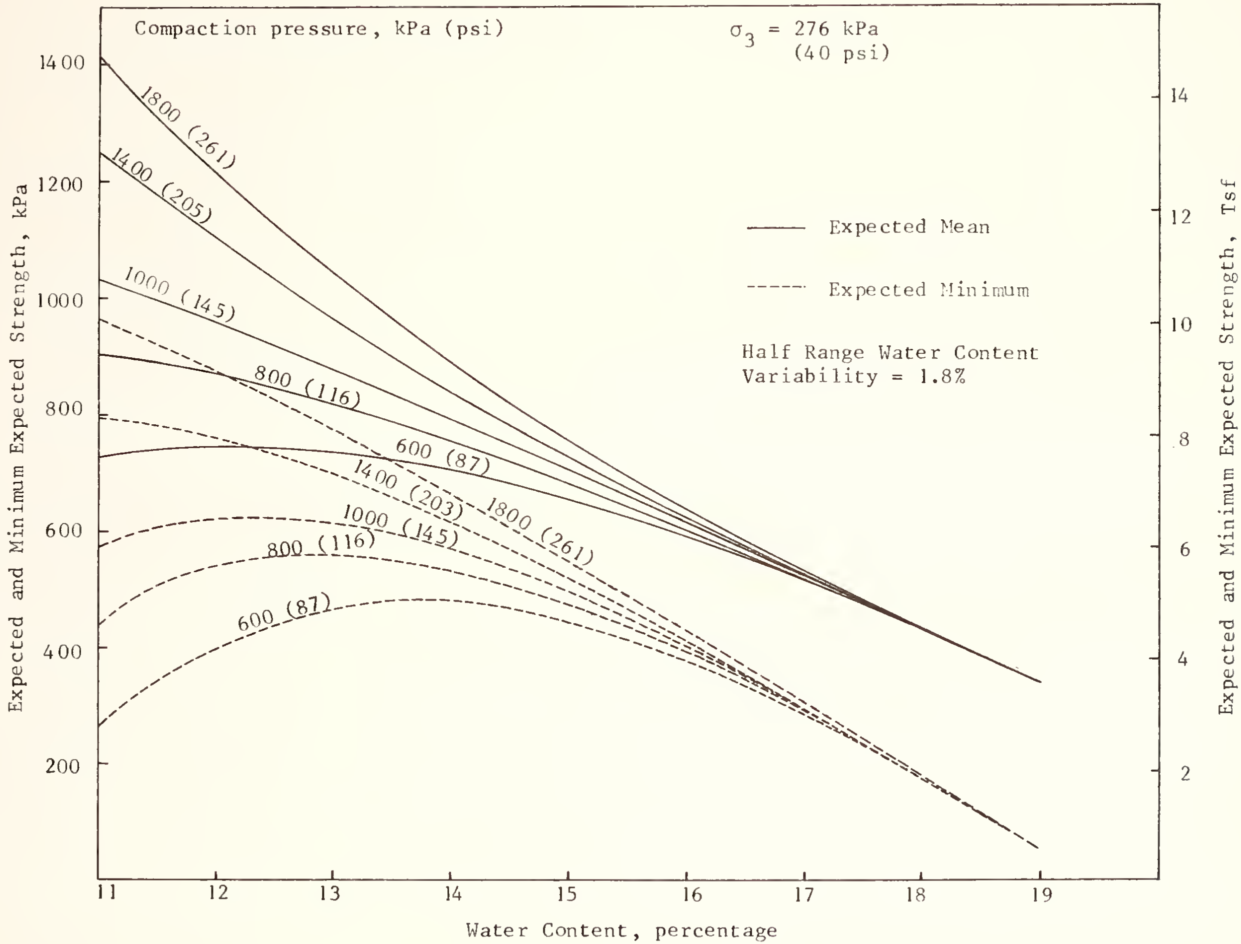

Figure 27 Design Chart for Field Confined Undrained Strength; σ_3 = 276 kPa (40 psi); (A-6, A-7-6)

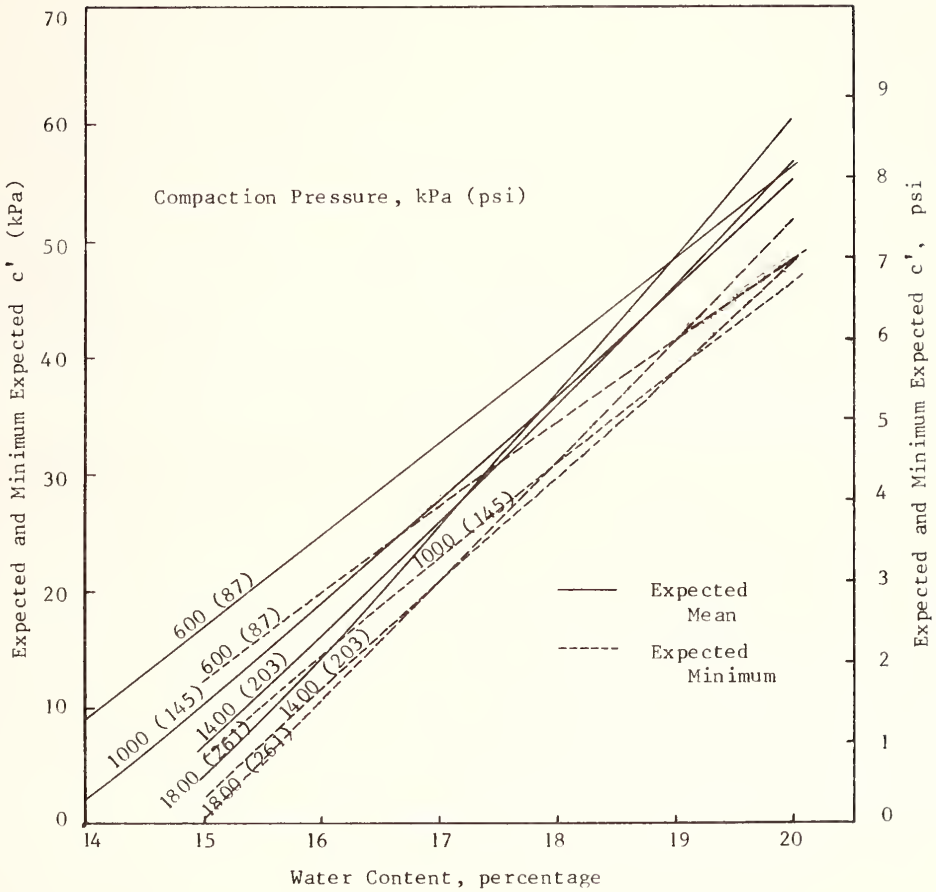

Figure 28 Design Chart for Field c'; (A-6, A-7-6)

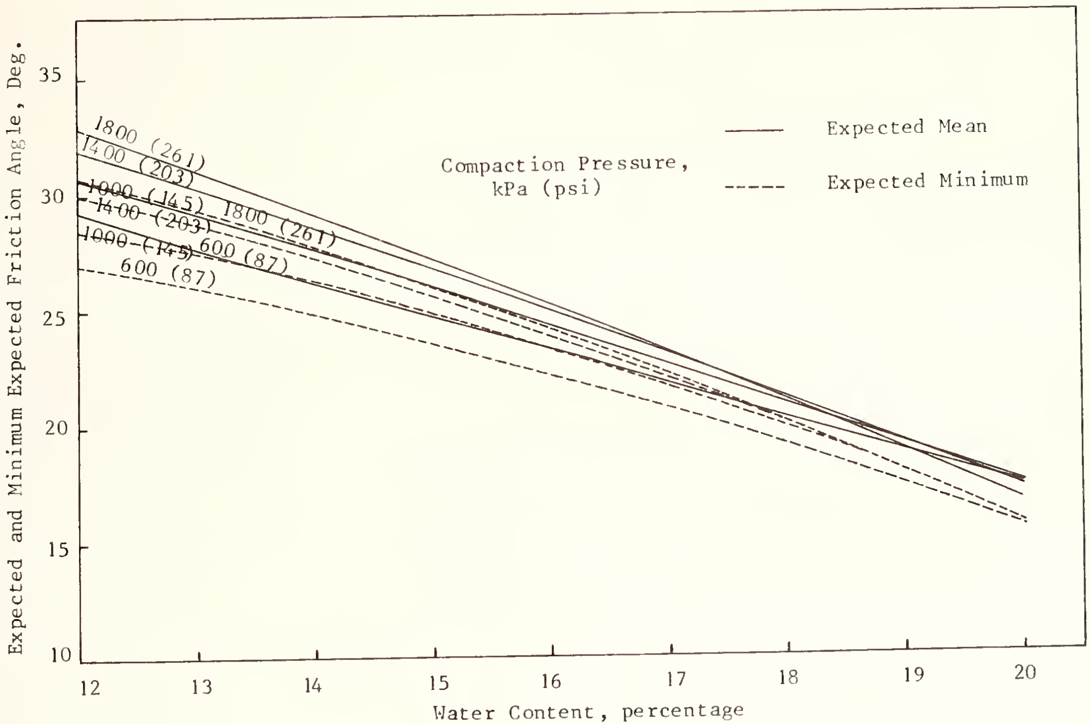

Figure 29 Design Chart for Field ϕ'; (A-6, A-7-6)

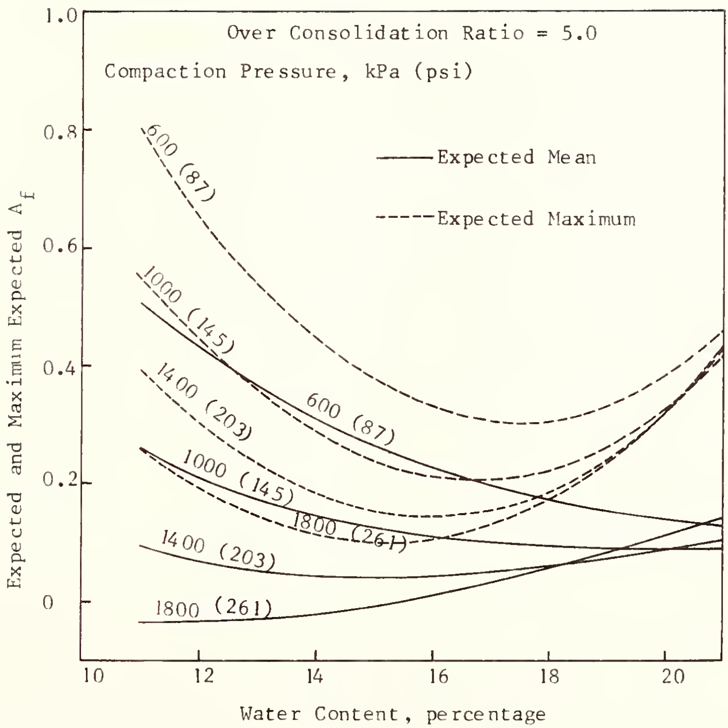

Figure 30 Design Chart for Field A_f; OCR = 5; (A-6, A-7-6)

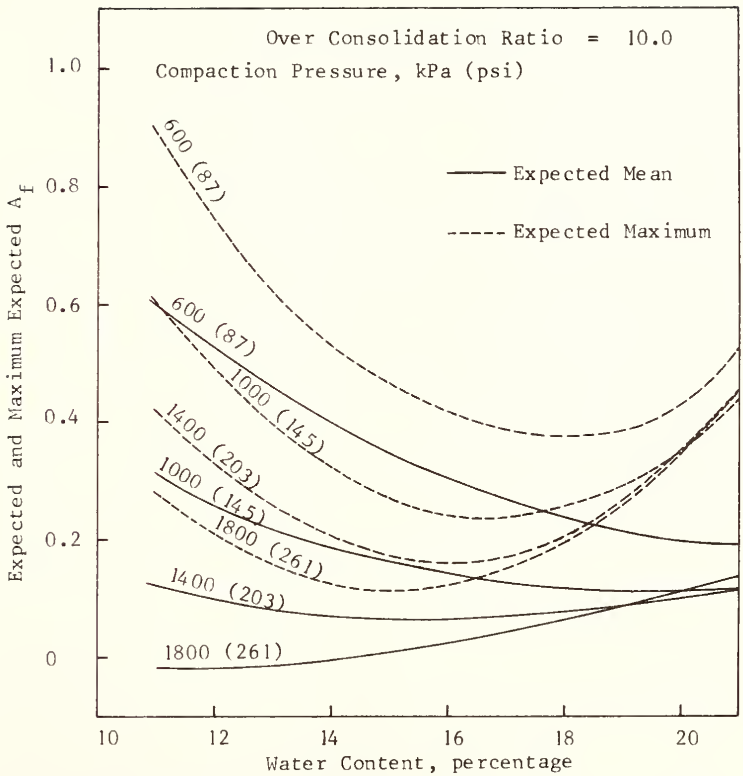

Figure 31 Design Chart for Field A_f; OCR = 10.; (A-6, A-7-6)

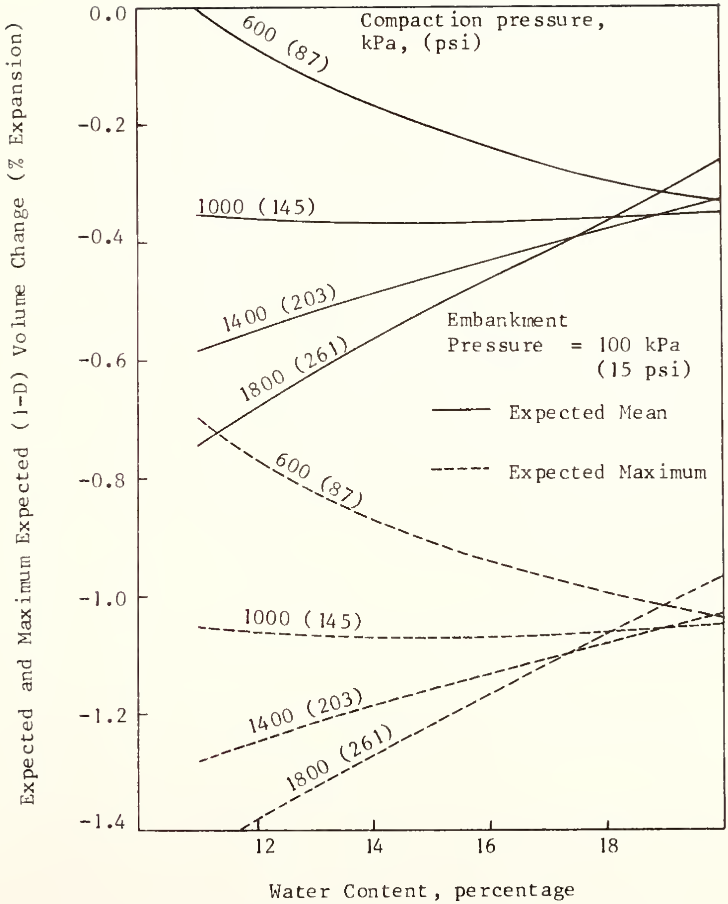

Figure 32 Design Chart for Field Volume Change; P_s = 100 kPa (15 psi); (A-6, A-7-6)

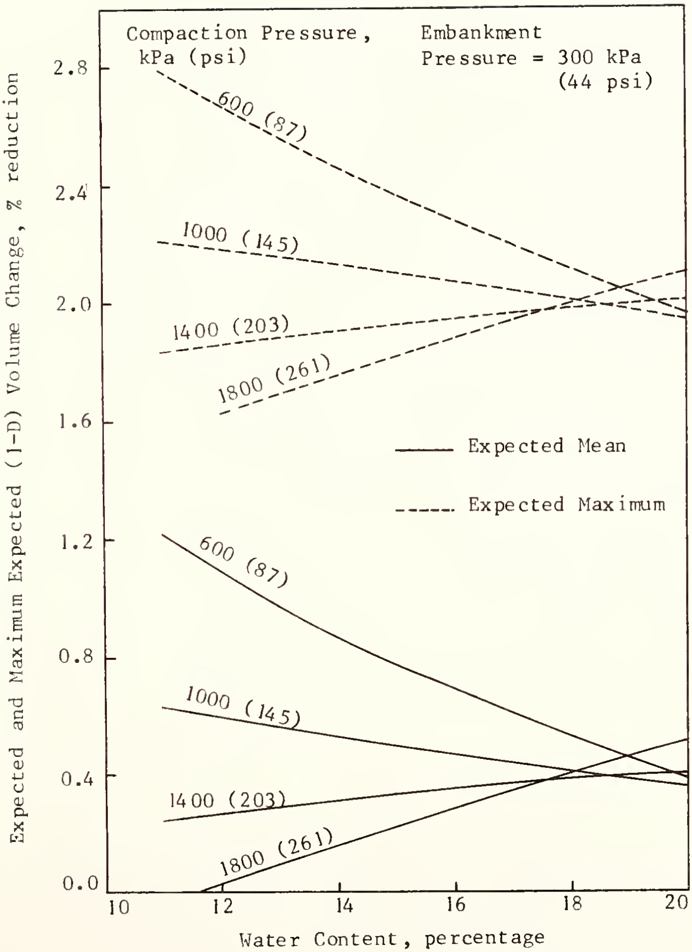

Figure 33 Design Chart for Field (1-D) Volume Change; P_s = 300 kPa (44 psi); (A-6, A-7-6)

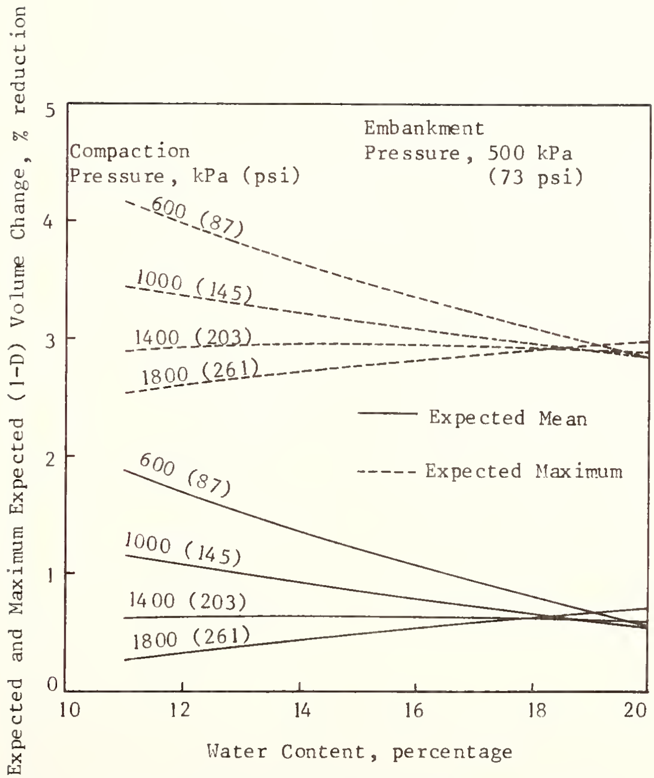

Figure 34 Design Chart for Field (1-D) Volume Change; P_s = 500 kPa (73 psi); (A-6, A-7-6)

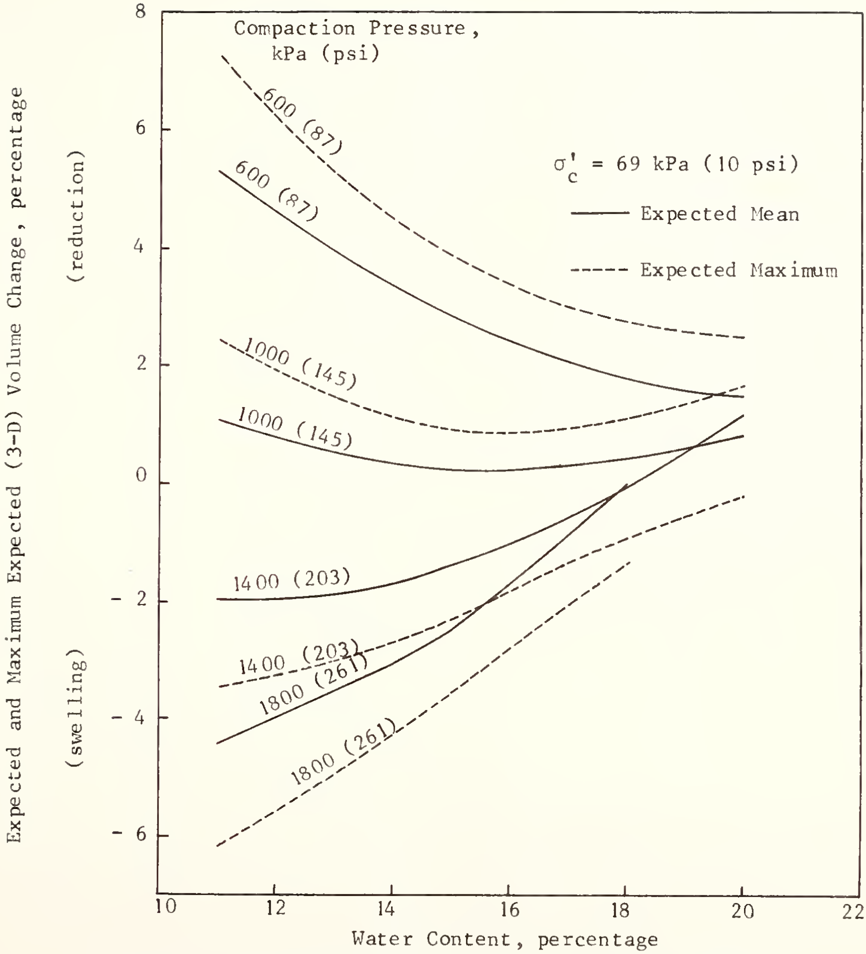

Figure 35 Design Chart for Field (3-D) Volume Change; σ'_c = 69 kPa (10 psi); (A-6, A-7-6)

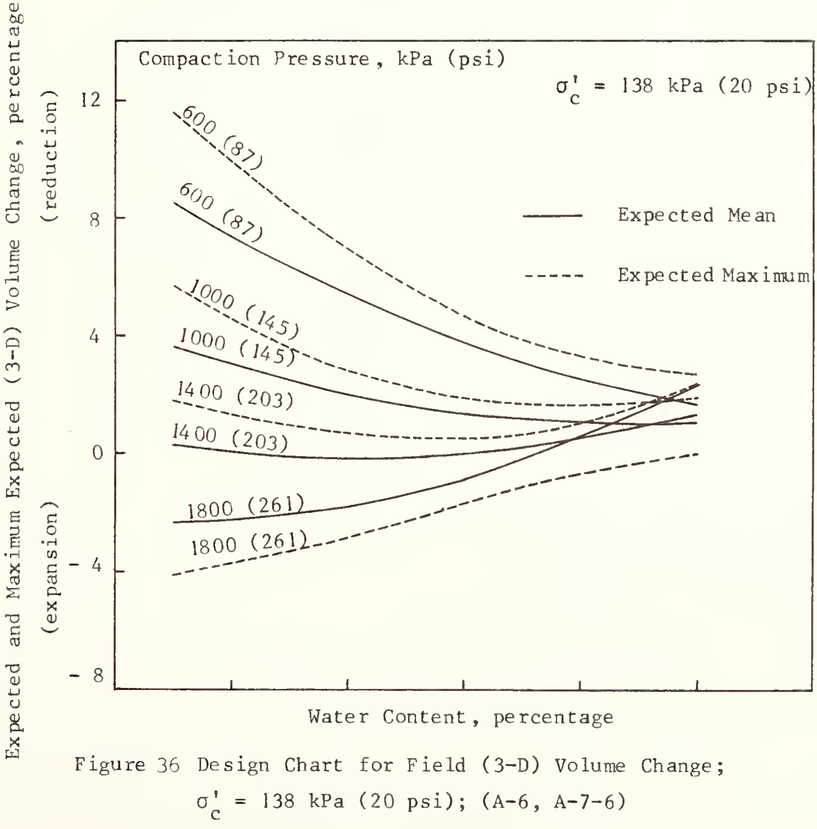

Figure 36 Design Chart for Field (3-D) Volume Change; σ_c' = 138 kPa (20 psi); (A-6, A-7-6)

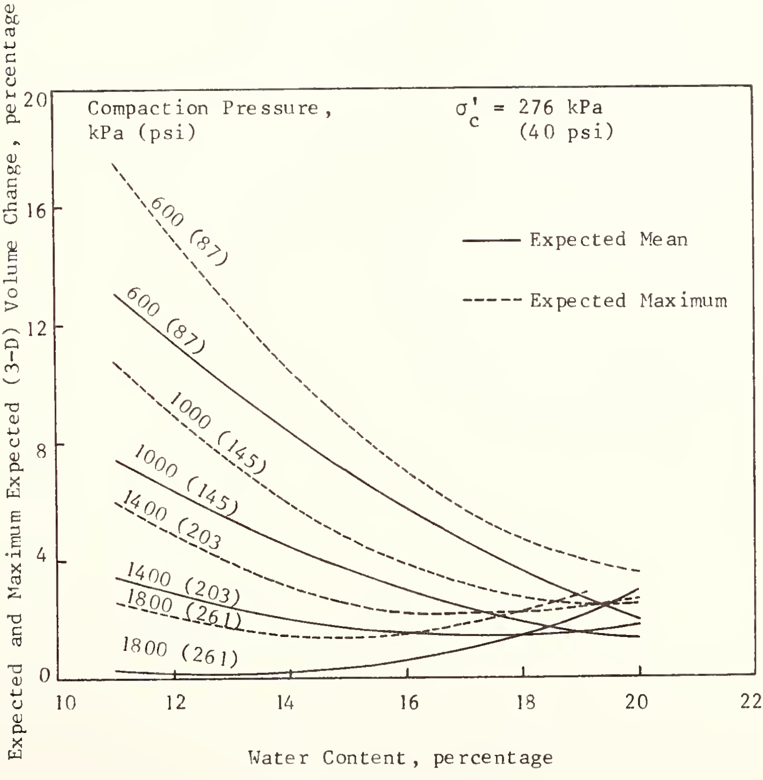

Figure 37 Design Chart for Field (3-D) Volume Change; σ'_c = 276 kPa (40 psi); (A-6, A-7-6)

Table 1
Relation Between Compaction Pressure and Roller Passes
(A-6, A-7-6 soil)

Roller	No. of Passes	Compaction Pressure (kPa) (psi)
Caterpillar	4	797 (115)
	8	1204 (174)
	16	1771 (257)
Rascal	4	780 (110)
	8	1038 (150)
	16	1525 (221)

Note: a loose lift thickness of 8 inches is assumed.

determine which assembly of parameters will give him the performance of his structure that he wants. Then, this array can be taken back to the charts to establish the appropriate specifications in terms of conventionally recognized compaction specifications.

To accomplish the foregoing requires that the borrow be identified prior to construction as well as the data base for a soil similar to the borrow and the equipment to be used in the project.

Where the borrow contains more than a single soil type, or where the data for the specific soil are not available in statistically significant quantity, the engineer will need to apply his best judgment to the values obtained from such equations or charts.

Quality Assurance

The creation of the specification to assure a desired upper or lower bound to the behavior parameters is not possible when the borrow is not identified prior to construction. It _is_ possible to use the data base from this study to allow prediction of the behavior of the compacted soil, as placed.

The identification characteristics of the soil used on the lift must first be obtained. Let us assume that the soil and roller are similar to those for which the data base is available. Then, the water content and dry density must be measured for at

least 7 samples from the lift (more, if possible); this will yield the mean water content and the range of water content in the lift as well as the mean dry density and its range. The number of roller passes must also be known by count.

The range in water content is entered into the program of Appendix A, and the output is used to prepare a chart similar to one shown as Table 2; the program will do all the calculations.

On this table, search column 1 to find the region corresponding to the lift mean water content; then find the line which corresponds to the lift mean water content and lift number of passes. Column 3 is the expected dry density (from the data base). Column 4 is the expected strength (from the data base). Colum 5 is the expected variability in dry density (from the data base); this is used to compare actually measured variability for the lift, as a check; if the comparison is poor, columns 6 and 8 may be in error. Column 6 is the expected variability in strength. Column 7 is the minimum expected dry density. Column 8 is the minimum expected strength, and this represents for the engineer the least magnitude of strength he should expect, with assurance.

Another example of such chart is shown as Table 3 (Terdich, p. 136) for swell pressure prediction. For this case, field optimum water content and maximum dry density must be obtained; this is done by determining the field compaction curve from a small test section or by using the procedures described in the

Table 2. Strength Variability Computer Program Output Representation

For Sheepsfoot Roller, A.C. - V(w) = 1.65 Percent A-4 Soil

Water Content (Percent)	Compactive Effort (Pass #)	Expected Dry Density (PCF)	Expected Strength (PSI)	Expected Dry Density Variability (PCF)	Expected Strength Variability (PSI)	Expected Min. Dry Density (PCF)	Expected Min. Strength (PSI)
11.00	9.00	118.5	41.30	2.28	10.00	115.78	31.30
12.00	9.00	116.65	38.75	1.91	8.64	114.74	30.11
13.00	9.00	115.26	36.20	1.57	7.38	113.69	28.82
14.00	9.00	113.86	33.65	1.25	6.30	112.60	27.35
15.00	9.00	112.46	31.10	1.21	6.19	111.25	24.19
16.00	9.00	111.06	28.54	1.51	7.36	109.55	21.18
17.00	9.00	109.66	25.99	1.86	8.70	107.81	17.30
18.00	9.00	108.27	23.44	2.22	10.11	106.05	13.33
11.00	10.00	118.05	41.88	2.28	10.03	115.78	31.86
12.00	10.00	116.65	39.33	1.91	8.69	114.74	30.64
13.00	10.00	115.26	36.78	1.57	7.48	113.69	29.31
14.00	10.00	113.86	34.23	1.25	6.46	112.60	27.78

Table 3 Swell Pressure Variability Computer Program Output Representation

$V(\Delta w)$ (%)	Δw (%)	No. of Passes	E_R	$\hat{\gamma}_d$ (pcf)	Field γ_p (pcr)	$V(\hat{\gamma}_d)$ (pcf)	SP (psi)	V(SP) (psi)	SP_{max} (psi)
1.4	-2.0	4.0	1.0	106.6	99.1	13.3	.7	13.3	13.9
1.4	-1.5	4.0	1.0	109.5	101.8	11.1	11.5	11.1	22.6
1.4	-1.0	4.0	1.0	112.4	104.6	8.9	22.4	8.9	31.3
1.4	- .5	4.0	1.0	115.4	107.3	6.7	33.3	6.7	39.9
1.4	0	4.0	1.0	118.3	110.0	8.7	44.1	8.7	52.8
1.4	0	4.0	1.0	115.3	107.2	1.3	43.1	9.4	52.4
1.4	.5	4.0	1.0	114.5	106.5	1.2	37.8	7.6	45.5
1.4	1.0	4.0	1.0	113.7	105.7	1.1	33.0	6.1	39.1
1.4	1.5	4.0	1.0	112.9	105.0	1.0	28.6	4.8	33.4
1.4	2.0	4.0	1.0	112.1	104.2	1.0	24.5	4.0	28.6
1.4	2.5	4.0	1.0	111.3	103.5	1.0	20.9	4.3	25.2
1.4	3.0	4.0	1.0	110.5	102.7	1.1	17.7	4.6	22.2
1.4	3.5	4.0	1.0	109.7	102.0	1.2	14.8	4.8	19.6
1.4	4.0	4.0	1.0	108.9	101.2	1.3	12.4	5.0	17.4
1.4	4.5	4.0	1.0	108.1	100.5	1.4	10.4	5.3	15.7
1.4	5.0	4.0	1.0	107.3	99.8	1.5	8.7	5.8	14.5
1.4	5.5	4.0	1.0	106.5	99.0	1.6	7.5	6.3	13.8
1.4	6.0	4.0	1.0	105.7	98.3	1.7	6.7	7.1	13.8

report of task FF-GG or E of this project; the result will be the predicted field max. γ_D and field OMC. The number of roller passes must be known, and water content and dry density of the lift must be determined from at least 7 samples randomly selected. Mean water content and mean dry density are calculated. Variability in water content and dry density are calculated as the half-ranges found for them from the samples. Then, ΔW is calculated as the difference between the mean water content and the field OMC determined previously. The measured half-range of water content is the data of column 1 of Table 3, and comes from the program of Appendix A, one input for which is the range of water content found on the lift. Then locate the proper line corresponding to that ΔW, number of passes, and energy ratio (E_r = number of passes/4).

Column 5 of Table is the expected mean dry density of the lift, and should correspond to corrected nuclear dry density determinations. Column 6 is expected dry density corresponding to uncorrected nuclear measurements. The actually measured mean dry density is compared with Column 5 or 6 as a check for compatibility of the data. Column 7 shows expected dry density variation. The expected swell pressure is found in column 8, and its expected variability is given in column 9. Column 10 gives the maximum swell pressure the engineer should expect in that lift, with assurance.

In each case, the engineer is reminded that the prediction equations and the charts developed therefrom require statistical

data for rigorous interpretation. When these are not available for the specific job, the equations and charts simply afford guidance for decisions which are largely judgmental.

Procedures for Using Charts for Cases of Other Similar Soils

It is likely that field projects will involve soils that are similar to those of this project, but not identical. The direct use of the charts, et al, of the project will create errors in the application.

For any contemplated use of the charts, first produce the laboratory Standard Proctor compaction curve. Compare the curve with that for this study's soil at the same energy level. Obtain a Translation Factor, TF[1],

$$TF = \frac{\text{OMC data base}}{\text{OMC new soil}}$$

where OMC = optimum water content

Then apply the TF to the (compaction) w and e_o in the regression equations for the parameters for the laboratory conditions.

The "translated" laboratory relations for parameters may then be compared with the curves from the data base of this project. Judgment could be applied to allow predictions, for example, to be made.

1. This approach is recommended in JHRP 82-1, page 199. An alternate idea for the effect of soil difference uses the ratio of Plasticity Indices (JHRP 81-14, page 123).

It is also possible to use correlations that have been made for several properties. These correlations are for the field parameter in terms of the corresponding laboratory data in this study. These may be used for the other similar soils because of the translation that is made to "fit" the new data to the laboratory data of this data base.

As examples, some field parameters may be obtained by using the following derived solutions:

(wet-of-optimum) $\rho_{d_{field}} = 188.3 + 0.872\ \rho_{d\ lab}$ (as translated)

$q_{c_{field}} = 174.92 + 0.916\ q_{c_{lab}}$ using translated w, e_o)

(3-D) $\frac{\Delta V}{V}_{field} = 1.886 + 0.495\ \frac{\Delta V}{V})_{lab}$ (using translated w, e_o)

$p_{s_{field}} = -303.67 + 1.323\ p_{s\ lab}$ (using translated w, e_o)

$A_{f_{field}} = 0.223 + 0.66\ A_{f\ lab}$ (using calculated p_s and translated w, e_o)

where ρ_d = dry density, k_g/m^3

q_c = confined undrained strength, kPa

$\frac{\Delta V}{V}$ = (3-D) volume change upon wetting, %

p_s = prestress, kPa

A_f = Skempton A-pore pressure coefficient

Prediction Validations

Several tube samples were obtained from project locations that were composed of similar soil(s) to that of the data base. It was intended to use the data base to predict what the behavior parameters should be, and then actually test for them. This was to be one check on the validity of the procedures suggested by this report.

The Table 4 shows the information for sample location as well as the other input available from inspection testing at time of construction. The Table 4 also shows the predicted and actual parameters obtained for the reader's comparison.

Table 4a
Sampling Location SR 446 near St. Croix

Compaction water content %	Compacted mass unit weight kg/m^3	Compaction pressure kPa	volume change $\Delta V/V_o$ % Predicted Confining Pressure, kPa				Measured Confining Pressure, kPa			
			69	138	207	241	69	138	207	241
8.5-12.1	1860-2057	780	1.66 to 5.67	3.77 to 9.9	5.4 to 13.22	6.09 to 14.06	0.50 to 0.58	2.98 to 3.97	3.7	6.05

Table 4a (Continued)
Sampling Location SR 446 near St. Croix

Compaction water content %	Compacted mass unit weight kg/m^3	Compaction pressure kPa	Effective stress strength intercept, c´ (kPa)		Effective stress friction angle ϕ (Deg)	
			Predicted	Measured	Predicted	Measured
8.5-12.1	1860-2057	780	0	0000	31-34	33-34

Table 4b
Sampling Location SR 37 near St. Croix, Indiana

Compaction water content %	Compacted mass unit weight kg/m^3	Compaction pressure kPa	volume change $\Delta V/V_o$ % — Predicted Confining Pressure, kPa					Measured Confining Pressure, kPa				
			34	69	138	207	270	34	69	138	207	270
18-20.5	1898-2203	700	0.29 to 2.6	-0.04 to 3.84	-0.51 to 5.56	-0.862 to 6.88	-1.16 to 7.99	1.65	2.28 to 4.13	4.17	7.4	7.08

4b (Continued)
Sampling Location SR 37 near St. Croix, Indiana

Compaction water content %	Compacted mass unit weight kg/m^3	Compaction pressure kPa	Effective stress strength intercept, c' (kPa)		Effect stress friction angle ϕ (Deg)	
			Predicted	Measured	Predicted	Measured
18-20.5	1898-2203	700	43.7 to 65	0 to 22	15.8 to 19.6	24.75 to 39.7*

Note * contained significant sandstone fragments

Table 4c
Sampling Location SR 37 near St. Croix, Indiana

Sample	Compaction Water Content %	Compacted mass unit weight kg/m^3	Optimum water content	Compaction pressure kPa	Embankment pressure, P_o kPa	1-D volume change % Measured	1-D volume change % Predicted
C1	17-20.5	1946-2041	20.0	573	381	1.43	0.796 to 1.48
C2	18-20.5	1898-2203	20.6	700	254	0.266	-0.19 to 1.3
					74.3	-0.413	-0.02 to -0.79
C3	18-20.5	1898-2203	20.6	700	1290	0.344	-1.33 to 2.48

Table 4c (Continued)
Sampling Location SR 37 near St. Croix, Indiana

Sample	Compaction Water Content %	Compacted mass unit weight kg/m^3	Optimum water content	Compaction pressure kPa	Embankment pressure, P_o kPa	Prestress kPa Measured	Prestress kPa Predicted	Swell Pressure kPa Measured	Swell Pressure kPa Predicted
C1	17-20.5	1946-2041	20.0	573	381	680	334 to 384	33	-145 to 379
C2	18-20.5	1898-2203	20.6	700	254 74.3	490	369-412	87	-85.5 to 289
C3	18-20.5	1898-2203	20.6	700	1290	1000	369-412	-	-

Implementation

The data base assembled in this project for the A-4 (glacial silty clay) and the A-6, A-7-6 (residual clay from shale) soils allows a significant capability to predict their in-service performance. It allows determining the actual magnitude of the parameters to be predicted from inspection test results. In addition, it allows a methodical procedure to create the compaction specification which will assure the engineer the presence, in-service, of a minimum or maximum magnitude of parameter(s), as he may desire. This is a giant improvement in such capability beyond the present state-of-the-art. The data base needs, and deserves, expansion.

Enlargement of the data base should be managed in several directions: (1) A-4 and A-6, A-7-6 soils of different geologic origin; (2) A-4 and A-6, A-7-6 soils compacted with different rollers; (3) different soils, especially clays of moderate to large plasticities.

We propose that the data base be enlarged for Indiana. We propose that Purdue University School of Civil Engineering act in cooperation with IDOH to methodically collect construction data on earthwork. In addition, selected projects will be sampled and the samples subjected to the largest possible array of testing to determine their field compacted behavior parameters. As the data accumulate to an appropriate magnitude, they will be assembled and manipulated to either 1) check the possible generalization of

this project's data base to soils near to those of this project; or 2) prepare charts and diagrams for the soils and rollers that appear to be associated with relations different from those of this data base.

In addition, there are additional charts and tables that can be produced from this project's data. These can be prepared, with the advice of the user IDOH to create a suite of charts ready for future use.

It is intended that these implementation plans begin on a cooperative venture basis. We propose Purdue should begin the assembly and testing. Then, if IDOH wishes to improve its own in-house programs, the work could shift to them. A 3-year period of effort is suggested to see how well collection and assembly had progressed in that time.

Finally, two additional areas of implementation should be emphasized: (1) collected data lend themselves to the formulation of a simplified phenomenological model for compacted clay behavior, such as is already available for saturated in-situ clays. (2) the laboratory test methods, statistical techniques, and translations of properties from laboratory to field are useful to the IDOH, and should be considered by this agency for incorporation into their practices.

```
            DEN1 = DEN + D
          ELSEIF (N.EQ.3) THEN
            XAS1 = XAH + DELTAW
            DEN1 = DEN + D
          ELSEIF (N.EQ.4) THEN
            XAS1 = XAH + DELTAW
            DEN1 = DEN - D
          ELSEIF (N.EQ.5) THEN
            XAS1 = XAH
            DEN1 = DEN
          ENDIF
C
C         THE FOLLOWING VARIABLES ARE THE TERMS IN THE REGRESSION
C         EQUATION FOR STRENGTH.  DEPENDING UPON THE EQUATION
C         USED THEY MAY BE CHANGED
C
           XS(2) = XAS1
           XS(3) = (XAS1-10.)**2
           XS(4) = (SQRT(DEN1*XAS1))*SQRT(SIGMA3)
           XS(5) = SQRT(DEN1)
           XS(6) = DEN1*XAS1
         IF (N.GT.4) GO TO 35
           SUM6 = 0.0
           DO 30 J=1,NS
           SUM5=0.0
           DO 40 I=1,NS
40         SUM5 = XS(I)*XXS(I,J)+SUM5
           SUM6 = SUM5*XS(J)+SUM6
30         CONTINUE
C
          V1 = SQRT(SUM6)
          VA(N) = LS*V1
          IF (VA(N).GE.VA(1)) VAS1 = VA(N)
 C        VAS1 = STRENGTH VARIABILITY
        GO TO 150
35       CONTINUE
          SUM3 = 0.0
          DO 125 I=1,NS
125       SUM3 = SUM3 + XS(I)*BETA(I)
          Q = SUM3
C         Q = VALUE OF STRENGTH CALCULATED FROM REGRESSION EQUATION
150     CONTINUE
          EMS = Q - VAS1
          EMD = DEN - D
C         EMS = EXPECTED MINIMUM STRENGTH
C         EMD = EXPECTED MINIMUM DRY DENSITY
        WRITE(6,1000) XAH,PC,DEN,Q,D,VAS1,EMS,EMD
1000    FORMAT(//10X,8(F8.2,2X))
         XAH = XAH + 1.0
105     CONTINUE
        STOP
        END
```

```
     1          T10,≠HALF RANGE WATER CONTENT = ≠,F5.2)
      WRITE(6,1001)
1001  FORMAT(////T10,≠WATER≠,T20,≠COMPACTION≠,T32,≠DENSITY≠,T40,
     1≠STRENGTH≠,T50,≠DENSITY≠,T60,≠STRENGTH≠,T70,≠MIN. EXP.≠,T80,
     2≠MIN. EXP.≠,/,T10,≠CONTENT≠,T20,≠PRESSURE≠,T50,≠VARIAB.≠,
     3T60,≠VARIAB.≠,T70,≠STRENGTH≠,T80,≠DENSITY≠//)
      XAH = 11.
C     XAH = WATER CONTENT
      DO 105  L  = 1,12
C
C     DRY DENSITY VARIABILITY CALCULATIONS BEGIN
C
      DO 110  N = 1,3
        IF (N.EQ.1) XAD1=XAH-DELTAW
        IF (N.EQ.2) XAD1=XAH+DELTAW
        IF (N.EQ.3) XAD1=XAH
C
C      THE FOLLOWING VARIABLES ARE THE TERMS IN THE REGRESSION EQUATION
C      FOR DRY DENSITY.  DEPENDING ON THE EQUATION USED THEY MAY BE
C      CHANGED.  SOME TERMS ARE EITHER MULTIPLIED OR DIVIDED BY
C      MULTIPLES OF 10 TO IMPROVE ACCRACY OF CALCULATIONS.
C
         XD(2) = (SQRT(PC))/XAD1
         XD(3) =((SQRT(PC))*XAD1**2)/100.
         XD(4) = XAD1*PC/100.
         XD(5) = 1000.0/XAD1
       IF (N.EQ.3) GO TO 110
         SUM2 = 0.0
         DO 20  J=1,ND
          SUM1=0.0
         DO 10  I=1,ND
10        SUM1 = XD(I)*XXD(I,J)+SUM1
20        SUM2 = SUM1*XD(J)+SUM2
           VDD = SQRT(SUM2)
           D1(N) = LDSS*VDD
           IF(D1(N).GE.D1(1)) D=D1(N)
C       D = DENSITY VARIABILITY
110    CONTINUE
C
C      DRY DENSITY VARIABILITY CALCULATIONS END
C
C      CALCULATION OF DENSITY TO USE IN STRENGTH EQUATION
C
       SUM7 = 0.0
       DO 115  I=1,ND
115    SUM7 = SUM7 + XD(I)*ALPHA(I)
       DEN = SUM7
N      DEN = DENSITY TO BE USED IN THE STRENGTH REGRESSION EQUATION
C
C      STRENGTH VARIABILITY IS CALCULATED AT FOUR POINTS AND THE
C      MAXIMUM VALUE IS TAKEN AS THE VARIABILITY AT THAT WATER CONTENT
C
        DO 150  N=1,5
         IF (N.EQ.1) THEN
           XAS1 = XAH - DELTAW
           DEN1 = DEN - D
         ELSEIF (N.EQ.2) THEN
           XAS1 = XAH - DELTAW
```

```
C     COMPUTER PROGRAM FOR DRY DENSITY AND STRENGTH VARIABILITY
C     CALCULATIONS FOR BOTH RASCAL AND CATERPILLAR
C
C     XD = VECTOR OF INDEPENDENT VARIABLES FOR DRY DENSITY
C     XXD = (X≠X)≠ FOR DRY DENSITY
C     LAMDAD = LAMDA FOR DRY DENSITY
C     SD = SQUARE ROOT OF THE MEAN SQUARE ERROR FOR DRY DENSITY
C     XS = VECTOR OF INDEPENDENT VARIABLES FOR STRENGTH
C     XXS = (X≠X)≠ FOR STRENGTH
C     LAMDAS = LAMDA FOR STRENGTH
C     SS = SQUARE ROOT OF THE MEAN SQUARE ERROR FOR STRENGTH
C     BETA = COEFFICIENTS OF THE STRENGTH REGRESSION EQUATION
C     ALPHA = COEFFICIENTS OF THE DENSITY REGRESSION EQUATION
C
      REAL LAMDAS,LAMDAD,LS,LDSS,D1(3),ALPHA(5),VA(6)
      PARAMETER(M1=6,N1=5)
      DIMENSION XS(M1),XXS(M1,M1),BETA(M1),XXD(N1,N1),XD(N1)
C
C     ND = NUMBER OF TERMS IN THE DRY DENSITY EQUATION
C     NS = NUMBER OF TERMS IN THE STRENGTH EQUATION
C
C     READ NESSASARY INFORMATION
C
      READ(5,*) ND,NS
      DO 1  I=1,ND
1     READ(5,*) (XXD(I,J),J=1,ND)
      READ(5,*) LAMDAD,SD
      READ(5,*) (ALPHA(I),I=1,5)
C
C
      DO 2  I=1,NS
2     READ(5,*) (XXS(I,J),J=1,NS)
      READ(5,*) LAMDAS,SS
      READ(5,*) (BETA(I),I=1,6)
      LS = LAMDAS*SS
      LDSS = LAMDAD*SD
C
C     INITIALIZE
C
      XD(1) = 1.0
      XS(1) = 1.0
C   XD(1) AND XS(1) ARE SET TO 1.0 IN ORDER TO ACCOMADATE
C   CONSTANT TERM OF THE REGRESSION EQUATION
C    ********************************************
C    *                COMPACTION PRESSURE (KPA)   *
C    * NO. OF PASSES ** CATERPILLAR ** RASCAL     *
C    ********************************************
C    *      4             797          780        *
C    *      8             1204         1038       *
C    *     16             1771         1525       *
C    ********************************************
C     SIGMA3 = CONFINING PRESSURE
C     VALUES USED ARE = 69,138,276
C     DELTAW = HALF RANGE WATER CONTENT VARIABILITY
C     PC = COMPACTION PRESSURE
      READ(5,*) SIGMA3,DELTAW,PC
      WRITE(6,55)  SIGMA3,DELTAW
55    FORMAT(///T10,≠CONFINING PRESSURE = ≠,F5.2,/,
```

Appendix B Summary of Regression Equations

A. Clay of Low Plasticity - (A-4) (from Task Phase I)

1) Laboratory - As-Compacted

Dependent Variable	Moisture Range	Regression Relationship	R^2
	All Moistures	No Models Significant	
Dry	Dry of Optimum	$\gamma_d = 0.375wE + 2.12w + 90.6$	0.905
Density	Wet of Optimum	$\gamma_d = -1.90w + 145.7$	0.919
	All Moistures	$q_u = 4.12\gamma_d - 0.049\gamma_d w - 393.3$	0.889
Unconfined	Dry of Optimum	$q_u = 4.14\gamma_d - 0.047\gamma_d w - 396.2$	0.888
Strength	Wet of Optimum	$q_u = 3.10\gamma_d - 0.073\gamma_d w - 231.8$	0.943

γ_D = p.c.f., dry unit weight

w = %, water content

E = energy, lb/ft^3, as a ratio to 7400 lb/ft^3

q_u = p.s.i., unconfined compression strength

A. Clay of Low Plasticity - (A-4) (from Task Phase I)

2) Laboratory - Soaked

Dependent Variable	Moisture Range	Regression Relationship	R^2
Dry Density	All Moistures	$\gamma_d = 17.257w-0.671w^2+9.962E-0.774wE+10.85$	0.657
	Dry of Optimum	$\gamma_d = 3.253w+3.334E+81.84$	0.811
	Wet of Optimum	$\gamma_d = 6.60E-1.099wE+0.00821w^2E^2+128.79$	0.519
Soaked Strength	All Moistures	$q_u^s = 26.184w-0.178\gamma_d^2+0.00106\gamma_d^3-0.000078w^2\gamma_d^2+591.79$	0.553
	Dry of Optimum	$q_u^s = -13.19E^2+0.000679\gamma_d^2E^2+0.0385w^2E^2+3.53$	0.715
	Wet of Optimum	$q_u^s = -4.71w+69.427$	0.605

γ_D = p.c.f., dry unit weight

w = %, water content

E = energy #/ft^3 as ratio to 7400 #/ft^3

q_u = p.s.i., unconfined compression strength

A. Clay of Low Plasticity - (A-4) (from Task EE)

3) Field

		R^2
a)	sheepsfoot roller - as-compacted	
	$\gamma_D = 133.43 - 1.398\ w$	0.58
	$q_u = 16.99 - 2.068\ w + 0.585\ E + 0.353\ \gamma_D$	0.42
b)	rubber-tired roller - as-compacted	
	$\gamma_D = 122.46 - 0.0476\ w^2 + 0.169\ E$	0.65
	$q_u = 51.91 - 2.224\ w - 0.155\ E + 0.0734\ \gamma_D$	0.29
c)	rubber-tired roller - soaked	
	$\gamma_D = 110.75 + 1.529\ E - 0.0958\ (w)(E)$	0.49
	$q_u = -118.37 + 1.189\ w + 0.195\ E + 0.968\ \gamma_D$	0.66

γ_D = p.c.f., dry unit weight

q_u = p.s.i., unconfined compression strength

w = %, water content

E = no. passes

B. Clay of Moderate Plasticity - (A-6, A-7-6) (from Task A)

1) Laboratory

a) prestress

$$p_s = -\ 343.13 - 0.0020\ (w^2)(p_c) + 48.91\ (p_c^{1/2}) \qquad 0.88$$

b) $$(1\text{-D})\frac{\Delta V}{V} = 24.57 - 0.872\ w - 0.0048\ p_c \qquad 0.86$$

p_s = prestress - kN/m^2

w = %, water content

p_c = compaction pressure, kPa

$\frac{\Delta V}{V}$ = (1-D) volume change, %, due to wetting

B. Clay of Moderate Plasticity - (A-6, A-7-6) (from Task C)

2) Laboratory

		R^2
a)	(1) dry of OMC $\rho_d = 1338.3 + 1284\ W_r^{1/2}/w + 0.32\ (w^2)(W_r^{1/2})$	0.99
	(2) wet of OMC $\rho_d = 961.8 + 15564/w$	0.99
b)	(1) dry of OMC $q_c = -1784.8 + 3.1\ (\rho_d)(S_i^{1/2}/w) - 84.0\ (1 - \frac{S_i}{100})(\sigma_3^{1/2})$	0.98
	(2) wet of OMC $q_c = 1.70\ (\rho_d)/(G_s\ \rho_w - \rho_d)$	0.99

ρ_d = dry density = kg/m^3

W_r = a work ratio in kneading compaction

L = 1.00 (simulated 0.6 Standard Proctor energy)

S = 1.88 (simulates Standard Proctor)

H = 9.09 (simulates Modified Proctor)

w = %, water content

S_i = %, initial degree of saturation

σ_3 = kN/m^2, confining pressure

q_c = kN/m^2, confined undrained compressive strength

B. Clay of Moderate Plasticity - (A-6, A-7-6) (from Task B)

3) Laboratory

a)	(3-D) $\frac{\Delta V}{V} = 28.48 - 1.36\ (10^{-5})(\rho_d)^2 + 0.0077\ (S_i)(\sigma'_c)^{1/2}$	0.95
b)	$A_f = 1.79 - 0.00011\ (\rho_d)(S_i)^{1/2} + 1.28\ (\sigma'_c)/\rho_d$	0.69
c)	$\phi' = 20.1^\circ \pm 1.3^\circ$	
d)	$c' = 1.71 - 3.83\ (w)(\log e_o)$	
	or use 10 kN/m^2 because range is small	

$\frac{\Delta V}{V}$ = 3-D volume change, %, due to wetting

ρ_d = dry density, kg/m^3

S_i = initial degree of saturation, %

σ'_c = isotropic consolidation pressure, kN/m^2

A_f = Skempton A-coefficient at failure

c' = effective stress strength intercept, kN/m^2

ϕ' = effective stress strength angle, degrees

B.4 Clay of Moderate Plasticity - (A-6, A-7-6) (from Table FF-GG)

Compactor Type	Conditions for Application	Regression Model	R^2
Caterpillar	Dry of OMC	$\hat{\gamma}_d = 5.863\ \Delta w + 118.3$	0.458
	Wet of OMC	$\hat{\gamma}_d = -1.609\ \Delta w + 1.006\ E_R + 114.3$	0.828
- Field	Dry of OMC	$SP = 21.72\ \Delta w + 44.12$	0.461
	Wet of OMC	$SP = -10.87\ \Delta w + 0.7998\ \Delta w^2 + 43.08$	0.695
Rascal	Dry of OMC	$\hat{\gamma}_d = 1.906\ \Delta w + 1.390\ E_R + 112.5$	0.609
	Wet of OMC	$\hat{\gamma}_d = -1.378\ \Delta w + 1.044\ E_R + 112.0$	0.723
- Field	A & B Energy Levels	$SP = 0.2919\ \Delta w + 4.593\ E_R - 0.02357\ \Delta w \gamma_d + 9.908$	0.260
	C Energy Level	$SP = -16.08\ \Delta w + 0.1501\ \Delta w \gamma_d + 14.50$	0.269
Impact	Dry of OMC	$\hat{\gamma}_d = 1.428\ \Delta w + 2.194\ E_R + 109.1$	0.943
	Wet of OMC	$\hat{\gamma}_d = -1.849\ \Delta w + 1.976\ E_R + 110.2$	0.928
- Laboratory	All Δw and E_R	$SP = 70.03\ \Delta w + 7.221\ E_R - 0.6821\ \Delta w \gamma_d + 3.531$	0.917
Kneading	Dry of OMC	$\hat{\gamma}_d = 2.034\ \Delta w + 2.126\ E_R + 109.9$	0.960
	Wet of OMC	$\hat{\gamma}_d = -2.190\ \Delta w + 2.164\ E_R + 111.0$	0.960
- Laboratory	All Δw and E_R	$SP = 40.29\ \Delta w + 8.018\ E_R - 0.3996\ \Delta w \gamma_d + 0.04564$	0.867

SP = swell pressure, psi

γ_D = dry unit weight, p.c.f.

Δw = %, water content difference from OMC

E_R = energy ratio, field = no. passes as ratio to 4

lab = ratio lb to 247.5 impact

ratio psi gage press. to 4

B.5 Clay of Moderate Plasticity - (A-6, A-7-6) (from Task D and recalculations for Task HH)

a) field $p_s = -160.99 - 0.00063\ (w^2)(p_c) + 27.04\ (p_c^{1/2})$ 0.86

b) field $\frac{\Delta V}{V} = -\ 2.26 + 0.400\ (e_o)(p_o^{1/2}) - 0.00026\ (w)(p_o)$ 0.78

(1) (1-D)

<u>or</u>

(2) field (1-D) $\frac{\Delta V}{V} = -\ 2.172 + 1128.6\ (p_o^{1/2}/\rho_d) - 0.4136\ (p_o^{1/2})$

$- 0.245\ (10^{-3})(w)(p_o)$ 0.78

(3) lab $\frac{\Delta V}{V}$, as a remanipulation of data from Task A,

(1-D) lab $\frac{\Delta V}{V} = -6.50 + 1.102\ (e_o)(p_o^{1/2}) - 0.00102\ (w)(p_o)$ 0.78

p_s = prestress, kN/m^2

w = %, water content

p_c = compaction pressure, kN/m^2

$\frac{\Delta V}{V}$ = (1-D) volume change, %, due to wetting

e_o = initial void ratio

p_o = equivalent embankment pressure at time of wetting, kN/m^2

ρ_d = dry density, kg/m^3.

B.6 Clay of Moderate Plasticity - (A-6, A-7-6) (from Task E and recalculations from Task HH)

R^2

a) (1) field $\rho_d = 1938.6 + 197.5\ (p_c^{1/2}/w)$

$+ 0.000654\ (p_c^{1/2})(w^2) - 0.0080\ (w)(p_c)$

$- 6628/w$ 0.74

(2) a recalculation of Task C laboratory data to create a similar model

lab $\rho_d = 1566.38 + 62.45\ (p_c^{1/2}/w)$

$+ 0.00214\ (p_c^{1/2})(w^2) + 0.0031\ (w)(p_c)$

$- 2617.4/w$ 0.92

b) (1) field $q_c = - 6980.05 + 636.21\ (w) - 0.155\ (\rho_d)(w)$

$- 8.3\ (w^2)+112.1(1-\frac{S_i}{100})(\sigma_3^{1/2})$

$+ 3.6\ (\rho_d)(S_i^{1/2}/w)$ 0.74

or

(2) field $q_c = - 33093 + 1056\ (w) - 4.516\ (w^2)$

$+ 0.139\ [\rho_d)(w)(\sigma_3)]^{1/2}$

$+ 800\ (\rho_d)^{1/2} - 0.568\ (w)\ (\rho_d)$ 0.70

(3) a recalculation of the laboratory data of Task C to create a model similar to (b)(1)

lab $q_c = - 1878.2 + 51.54\ (w) - 0.06\ (\rho_d)(w)$

$+ 1.39\ (w^2) + 76.91\ (1 - \frac{S_i}{100})(\sigma_3^{1/2})$

$+ 3.68\ (\rho_d)(S_i^{1/2}/w)$ 0.98

c) (1) field (3-D) $\frac{\Delta V}{V} = - 0.166 + 2.47\ (e_o)(\sigma_c)^{1/2} - 0.365\ p_s$

$- 0.00263\ (w)(\sigma_c)$ 0.70

or

(2) field (3-D) $\frac{\Delta V}{V} = 9.13 + 5745\ (\sigma'_c)^{1/2}/\rho_d$
$+ 0.207\ (10^{-4})(w^2)(p_c) - 0.724\ [(w)(\sigma'_c)]^{1/2}$
$- 0.5344\ (p_c^{1/2})$ 0.73

(3) a recalculation of the laboratory data of Task B to create a model similar to (C)(1)

lab (3-D) $\frac{\Delta V}{V} = -\ 9.4 + 2.9\ (e_o)(\sigma'_c)^{1/2} - 0.404\ p_s$
$- 0.00276\ (w)(\sigma'_c)$ 0.95

d) (1) field $A_f = 2.05 - 0.73/e_o - 0.232\ (10^{-4})(\rho_d)(S_i)^{1/2}$
$- 0.382\ (\log OCR)$ 0.61

or

(2) field $A_f = 7.45 - 0.189\ (\rho_d) + 8.55/w$
$+ 12.73\ (OCR/p_c) + 0.00355\ (w/OCR)^2$ 0.69

(3) a recalculation of the laboratory data of Task B to create a model similar to (d)(1)

lab $A_f = 2.34 + 0.56/e_o - 0.189\ (10^{-3})(\rho_d)(S_i)^{1/2}$
$- 0.246\ (\log OCR)$ 0.70

e) (1) field $c' = -\ 102.79 + 11.208\ (w) + 14.55\ (w)(\log e_o$ 0.97

or

(2) field $c' = -\ 102.79 + 42.5\ (w) - 0.826\ (w)(\rho_d)^{1/2}$ 0.97

(3) field $\phi' = 47.56 - 2.112\ (w) - 2.625\ (w)(\log e_o)$ 0.88

or

(4) field $\phi' = 47.56 - 7.82\ (w) + 0.151\ (w)(\rho_d)^{1/2}$ 0.88

ρ_d = dry density, kg/m^3

p_c = compaction pressure, kPa

w = water content, %

q_c = confined undrained strength (i.e., stress difference at failure) kPa

S_i = initial degree of saturation, %

σ_3 = confining pressure, kPa

e_o = initial void ratio

σ'_c = isotropic consolidation pressure, kPa

p_s = prestress, kPa

OCR = p_s/σ'_c

log = base 10 logarithm

Appendix C - Adequacy of Drive Sampling Methods

Background

There were two test pads constructed for this project, and each was sampled to obtain specimens suitable for laboratory testing. Drive sample techniques were used in the first test pad. This involved driving 2.51-inch (6.38 cm) internal diameter tube having a wall thickness of 0.12 inches (0.30 cm) and a length of 5.0 inches (12.7 cm). A drop hammer of about 16.7 lbs (7.6 kg) weight was allowed to fall 28 inches (71 cm) upon a cushion block of wood to force the tube into the soil. Several blows (10 to 25) were required to drive each tube (about 40 ft.-lbs. (54 N•m)) of energy were applied with each blow. The tubes were usually sealed and transported, and the samples were extruded prior to testing.

Questions were asked about the extent of disturbance caused in the specimens by this seemingly harsh driving operation. Thus, sampling of the second test pad was designed with a group of comparison samples to also be obtained by hydraulic push techniques of the same type tubes. The second test pad was laid out as an assembly of 10 individual pads separated from each other. Each pad was intended to be prepared to a different specific initial water content and compaction was specified for one specific roller only. Each pad was divided into 2 ft by 2 ft grid (0.6 m by 0.6 m) for its entire 14 ft. (4.3 m) width and 116 ft (35.4 m) length. Locations of drive samples to be taken were determined

by random number techniques, while push samples were restricted to grid locations near the outer edges of the pads to accommodate the truck-mounted hydraulic ram. For this comparison study only samples from the Rascal-rolled pads were used. Each sample was tagged and labeled; for example Drive R1A3 meant Drive sample, Rascal compactor, 1st water content level, A energy level, 3rd sample taken for those compaction conditions. The R indicated the Rascal roller. There were 5 water content levels, intended to cover both wet and dry-side conditions. There were 3 energy levels, A, B, C to indicate 4, 8, 16 passes of the roller, respectively. Six samples were taken for each condition, i.e., 6 samples for R1A, etc. by drive sampling and 6 by push sampling.

The soil for the test pad was a residual clay from shale. The pad was constructed under presumably controlled conditions by which there was sizable mixing on the lift by discing. There was time allotted for added water to be distributed internally, and there was a plastic cover applied over the lift to help reduce evaporation. In spite of these effects several observations are pertinent: 1) the temperature each day exceeded 90^{o}F; 2) 4 to 6 hours were required to sample each pad completely; 3) rock fragments appeared to be present in almost all samples - in some cases the samples were valueless because they contained large fragments; 4) some outer surfaces of samples were badly gouged by fragments dragged by the tubes; 5) many dry-side samples were very brittle; 6) some samples exhibited horizontal cracks - this could have been caused either by compaction or sampling; 7) some

samples had inclusions of lumps of soil that had not been broken up and intermixed by the process; 8) some samples contained zones of distinctly different water contents.

The foregoing observations apply to samples obtained by both push and drive methods. The result is a large variability in water contents, the measured values of which are reported below for the pre-rolling condition as obtained by nuclear devices, and for conditions after compaction from nuclear devices as well as from the tested samples.

The data obtained by testing the samples that were intact and which could be prepared properly for testing are shown below.

There was no apparent relation between water content and strength for any one of the compaction energy levels. The strength data for each compaction energy level was then lumped by method of sampling, and an average was calculated; the results were:

Compaction Level A - avg. $q_{u_{push}}$ = 2.1 Tsf

avg. $q_{u_{drive}}$ = 2.1 Tsf

Compaction Level B - avg. $q_{u_{push}}$ = 3.5 Tsf

avg. $q_{u_{drive}}$ = 3.6 Tsf

Compaction Level C - avg. $q_{u_{push}}$ = 3.3 Tsf

avg. $q_{u_{drive}}$ = 3.3 Tsf

Table 5

Roller Water Content Level		Magnitudes of Pad Water Content (%)		
	Before Rolling	After 4 passes	After 8 passes	After 16 passes
R 1				
nuclear	not avail.	15.5 (14.6-16.4)	15.3 (13.9-16.8)	14.2 (13.6-14.7)
sample	-	14.4 (11.9-17.0)	15.0 (12.6-17.9)	15.2 (12.5-18.4)
R 2				
nuclear	17.1 (16.6-17.6)	15.5 (14.2-17.7)	14.2 (13.5-14.9)	13.6 (13.1-14.2)
sample	-	14.0 (11.2-17.4)	14.6 (9.4-16.2)	15.4 (12.4-19.9)
R 3				
nuclear	19.4 (18.2-21.3)	13.8 (13.0-14.6)	15.5 (12.9-17.0)	15.8 (15.0-16.5)
sample	-	15.9 (13.3-18.1)	14.0 (12.1-17.4)	14.1 (12.1-16.2)
R 4				
nuclear	15.8 (14.3-17.0)	20.2 (19.2-21.2)	21.8 (17.9-25.7)	19.9 (17.0-22.8)
sample	-	14.8 (9.4-18.1)	14.9 (13.1-16.3)	14.5 (14.0-15.2)
R 5				
nuclear	20.9 (18.7-23.5)	24.2 (22.5-26.0)	21.4 (19.2-23.6)	22.5 (17.0-28.1)
sample	-	15.3 (14.6-15.9)	17.4 (14.3-21.3)	17.1 (13.5-20.0)

Table 6 Strength Data From Rush and Drive Samples

Sample No.	Type	$w_{initial}$ (%)	$w_{failure}$ (%)	zone q_u (Tsf)	Remarks
R1A1	P	13.3			too short
	D	15.6			crumbly
R1A2	P	14.7			crumbly
	D	13.2			crumbly
R1A3	P	16.1			crumbly
	D	15.1			crumbly
R1A4	P	-			missing
	D	14.9			crumbly
R1A4	P	-			missing
	D	17.0			crumbly
R1A6	P	11.9			crumbly
	D	12.1			crumbly
R1B1	P	13.3			crumbly
	D	-			bent tube
R1B2	P	14.3	14.3	4.3	-
	D	15.3			broken in middle
R1B3	P	15.3			crumbly
	D	14.7			broken
R1B4	P	15.4			broken
	D	17.9			broken
R1B5	P	15.7	19.7	3.4	
	D	15.3	18.7	3.0	
R1B6	P	12.6	15.7	4.0	
	D	-	15.2	4.2	

For R1B

avg $q_{u_{push}}$ = 3 Tsf

avg $q_{u_{drive}}$ = 3.6 Tsf

R1C1	P	-			bent tube	
	D	12.5	11.4	8.2		
R1C2	P	14.8			sand seams	
	D	17.0			too short	
R1C3	P	17.0	15.8	2.8		
	D	15.2	12.5	1.5		For R1C
R1C4	P	13.4	18.3	4.5		avg. $q_{u_{push}}$ = 4.1 Tsf
	D	15.6			too short	avg. $q_{u_{drive}}$ = 4.9 Tsf
R1C5		15.9			crumbly	
	D	18.4			too short	
R1C6	P	13.2	13.8	5.0		
	D	13.6			crumbly	
R2A1	P	15.1	13.0	3.4		
	D	13.6	14.6	1.2		
R2A2	P	-			missing	
	D	-			missing	
R2A3	P				crumbly	
	D	15.4	15.5	3.1		For R2A
R2A4	P	17.4			crumbly	$q_{u_{push}}$ = 3.4 Tsf
	D	-			missing	avg. $q_{u_{drive}}$ = 2.2 Tsf
R2A5	P	11.2			large stone	
	D	12.2			crumbly	
R2A6	P	13.4			crumbly	
	D	11.8			crumbly	

R2B1	P	15.9	15.5	2.3		
	D	15.6	15.6	5.0		
R2B2	P	14.7			too short	
	D	14.4			too short	
R2B3	P	16.2			crumbly	For R2B
	D	16.2	12.8	3.3		avg. $q_{u_{push}} = 2.9$ Tsf
R2B4	P	14.5	9.0	3.4		avg. $q_{u_{drive}} = 4.2$ Tsf
	D	15.3			too brittle	
R2B5	P	13.3			crumbly	
	D	14.4			too short	
R2B6	P	-			bent tube	
	D	9.4			too short	
R2C1	P	14.5			crumbly	
	D	14.5			crumbly	
R2C2	P	15.9			too brittle	
	D	15.5			too short	
R2C3	P	15.1			crumbly	
	D	-	16.2	2.4		For R2C
R2C4	P	12.4			too brittle	avg. $q_{u_{push}} = 3.7$ Tsf
	D	13.5			too brittle	avg. $q_{u_{drive}} = 3.8$ Tsf
R2C5	P	-	16.0	3.7		
	D	19.9			too brittle	
R2C6	P	-	16.1	3.6		
	D	-	16.3	5.2		

R3A1	P	-			crumbled
	D	-			crumbled
R3A2	P	13.3			sample broke
	D	14.9			broken
R3A3	P	15.4			broken
	D	18.0			too short
R3A4	P	16.8			crumbly
	D	18.1			crumbly
R3A5	P	15.7			crumbly
	D	-			broken
R3A6	P	15.1			crumbly
	D	16.1			too short
R3B1	P	13.1			crumbly
	D	-			too short
R3B2	P	12.9			crumbly
	D	14.8			crumbly
R3B3	P	13.8			crumbly
	D	16.3			crumbly
R3B4	P	17.4	15.2	2.2	
	D	16.1			crumbly
R3B5	P	12.4			brittle
	D	12.5			brittle
R3B6	P	12.5			broken
	D	12.1			too short

R3C1	P	13.6	15.3	4.5	
	D	14.8			broken
R3C2	P	12.9			broken
	D	14.7			broken
R3C3	P	13.2			too strong
	D	12.1			broken
R3C4	P	13.5			broken
	D	14.5			too short
R3C5	P	14.7			too brittle
	D	16.2			broken
R3C6	P	14.4			broken
	D	14.2			too brittle
R4A1	P	13.7	17.1	3.7	
	D	17.3			crumbly
R4A2	P	17.6			broken
	D	17.8			large void
R4A3	P	18.1			broken
	D	16.1			broken
R4A4	P	16.2			too short
	D	-			large voids
R4A5	P	12.7			crumbly
	D	14.9			crumbly
R4A6	P	9.4			broken
	D	9.4			too short

R4B1	P	13.3	14.6	5.2		
	D	16.2	14.9	3.7		
R4B2	P	13.5			broken	
	D	13.7			broken	
R4B3	P	15.3			broken	For R4B
	D	13.1			too stony	avg. $q_{u_{push}}$ = 4.2 Tsf
R4B4	P	16.0	20.3	3.3		avg. $q_{u_{drive}}$ = 3.6 Tsf
	D	14.7	20.0	3.4		
R4B5	P	14.5	17.0	4.2		
	D	13.8			too stony	
R4B6	P	18.8			large stone	
	D	16.3			broken	
R4C1	P	14.0			crumbly	
	D	14.3			too short	
R4C2	P	15.2	20.0	5.1		
	D	-	16.9	4.5		
R4C3	P	-			missing	
	D	-	17.2	4.1		For R4C
R4C4	P	-	15.1	3.1		avg. $q_{u_{push}}$ = 4.1 Tsf
	D	-	25.8	2.1		avg. $q_{u_{drive}}$ = 3.6 Tsf
R4C5	P	-			missing	
	D	-	18.1	3.7		
R4C6	P	-			missing	
	D	-	18.4	3.4		

R5A1	P	15.9			too short	
	D	-			missing	
R5A2	P	-	21.6	1.5		
	D	-	18.6	1.5		
R5A3	P	-	-	3.0		
	D	-			missing	For R5A
R5A4	P	-	12.6	1.2		avg. $q_{u_{push}}$ = 1.5 Tsf
	D	-	17.0	1.9		avg. $q_{u_{drive}}$ = 2.1 Tsf
R5A5	P	14.6	13.7	0.9		
	D	-	15.4	3.0		
R5A6	P	-	17.8	1.0		
	D	-			too brittle	
R5B1	P	-	16.3	3.9		
	D	17.9			too short	
R5B2	P	19.5	19.3	2.0		
	D	21.3			too stony	
R5B3	P	-	22.9	2.7		
	D	15.7	20.9	2.8		For R5B
R5B4	P	14.9	16.2	3.1		avg. $q_{u_{push}}$ = 3.3 Tsf
	D	17.7	19.7	2.3		avg. $q_{u_{drive}}$ = 3.2 Tsf
R5B5	P	-	12.9	5.3		
	D	14.3	13.7	4.5		
R5B6	P	-	21.4	3.0		
	D	17.8			too brittle	
		.8				

		.8		
R5C1	P	15.8	-	2.9
	D	19.1	17.2	3.4
R5C2	P	-	21.3	1.3
	D	-	24.6	1.0
R5C3	P	20.0	19.4	2.0
	D	18.5	19.8	1.8
R5C4	P	14.9	13.0	3.3
	D	13.4	17.2	4.0
R5C5	P	-	21.3	1.6
	D	19.0	19.1	2.5
R5C6	P	17.4	21.4	2.1
	D	15.6	18.3	2.3
		.6	.3	.3

For R5C

avg. ${q_u}_{push} = 2.2$ Tsf

avg. ${q_u}_{drive} = 2.5$ Tsf

The conclusion was drawn that there was no significant difference in results of testing samples obtained by push methods or by drive methods.

Appendix D: Provisions for Test Pad Construction

Introduction

This appendix presents recommendations for the contents of special provisions that are written to facilitate proper test pad construction. The purpose of the test pads is to create compacted earth in uniform condition. The test pads will be sampled, and the samples will be tested; the results will be used to enlarge the capability of the engineer to control and predict the behavior of the field compacted soil in-service in general usage in other projects.

Test Pad Configuration

1. One test pad should be used for one roller operating on one soil at one controlled water content.

2. A total of 5 test pads are recommended to define the characteristics for one roller operating on one soil, each pad being at a different controlled water content.

3. Each test pad should be about 14 feet wide and about 115 feet long.

4. Test pads should be placed relative to each other to allow 10 feet between adjacent test pads and enough room between the ends of adjacent test pads to allow rollers to turn and reverse direction while totally clear of the test pads.

5. Along the outer sides of the test pads, wooden stakes should be set to allow the establishment of a series of grid lines on the test pad. The grid sections should be about 2 ft x 2 ft in area. These identified grids become locations for sampling, as established by a random number selection process.

6. The test pads can be constructed in any configuration with respect to each other to conform to project Right-of-Way constraints; they should, however, be close to each other.

Construction

1. The site of a test pad should be cleared of vegetation and other organic matter.

2. Construct a minimum thickness of 48 inches of prepared foundation; this may contain up to 24 inches of scarified and compacted subgrade; it must contain at least 24 inches of compacted fill using the selected borrow. Provisions of IDOH Standard Specifications should be used. The surface so created should be proof-rolled to remove soft locations.

Then, a 12-inch thick "sub-base" layer should be constructed using the same provisions as (2).

4. The test lift will then be placed. The loose soil should be distributed as evenly as possible over the entire

pad section to a loose depth of about 9 inches. The water content for the pad will be selected by the engineer, and the amount of water to be added will be directed by the engineer and controlled by him by suitable measurements. This water will be added by suitable means, preferably one that will not compact the lift.

5. The entire lift will be disked, blended, or otherwise mixed to distribute the water and to remove any compaction effects created during preparation. A plastic film will then be placed on top of the loose lift to reduce water losses.

6. For 3 or 4 days (as deemed necessary by the engineer), the plastic film will be removed, the loose lift will be disked thoroughly, and the film replaced each time. Water content determinations will be made to help decide when the water content in the lift is at the proper amount.

7. The roller will then be applied to the lift to produce a uniform coverage over the entire pad. All turning and positioning is to be done outside pad area. The same uniform speed for roller operation is to be used on the pad at all times. When the indicated number of passes has been applied, the roller will move off the pad.

8. Sampling operations should be performed by engineer personnel associated with IDOH.

9. After a round of sampling, additional passes of the roller will be directed by the engineer; these will be applied as in (7). Sampling will again follow.

10. The number of iterations of rolling and subsequent sampling will be determined by the engineer.

Appendix E

References: Reports Prepared for the Project

1. Task Phase I: Peterson, J. L., "Improving Embankment Design and Performance: Prediction of As-Compacted Field Strength by Laboratory Simulation", Report No. JHRP 75-22, Joint Highway Research Project, Purdue University, December 1975.

2. Task Phase I: Essigmann, M. F., Jr., "An Examination of the Variability Resulting from Soil Compaction", Report No. JHRP 76-28, Joint Highway Research Project, Purdue University, October 1976.

3. Task Phase I: Scott, J. C., "Examination of the Variability of the Soaked Strength of a Laboratory Compacted Clay", Report No. JHRP 77-8, Joint Highway Research Project, Purdue University, May 1977.

4. Task EE: Price, J. T., "Soil Compaction Specification Procedure for Desired Field Strength Response", Report No. JHRP 78-7, Joint Highway Research Project, Purdue University, June 1978.

5. Task A: DiBernardo, A. and Lovell, C. W., "The Effect of Laboratory Compaction on the Compressibility of a Highly Plastic Clay", Report No. FHWA/IN/JHRP-79/3, Joint Highway Research Project, Purdue University, May 1979.

6. Task B: Johnson, J. M. and Lovell, C. W., "The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation", Report No. FHWA/IN/JHRP-79/7, Joint Highway Research Project, Purdue University, August 1979.

7. Task C: Weitzel, D. W. and Lovell, C. W., "The Effect of Laboratory Compaction on the Unconsolidated-Undrained Strength Behavior of a Highly Plastic Clay", Report No. FHWA/IN/JHRP-79/11, Joint Highway Research Project, Purdue University, September 1979.

8. Tasks FF-GG: Terdich, G. M., "Prediction and Control of Field Swell Pressures of Compacted Medium Plastic Clay", Report No. FHWA/IN/JHRP-80/4, Joint Highway Research Project, Purdue University, March 1981.

9. Task D: Lin, P. S. and Lovell, C. W., "Compressibility of Field Compacted Clay", Report No. FHWA/IN/JHRP-81/14, Joint Highway Research Project, Purdue University, August 1981.

10. Task E: Liang, Y. and Lovell, C. W., "Strength of Field Compacted Clayey Embankments", Report No. FHWA/IN/JHRP-82/1, Joint Highway Research Project, Purdue University, February, 1982.

11. Task FFF: Goodman, M., Chameau, J. L., and Lovell, C. W., "Design of Compacted Clay Embankments for Improved Stability and Settlement Performance", Report No. FHWA/IN/JHRP-83/12.

References: Papers from Project Results

Abeyesekera, R. A. and Lovell, C. W. (1981). "Volume Changes in Compacted Clays and Shales Upon Saturation", Transportation Research Record 790, July. pp. 67-73.

DiBernardo, A. and Lovell, C. W. (1980). "Dependence of Compacted Clay Compressibility on Compaction Variables", Transportation Research Record 754, Transportation Research Board, Wash., D.C. Nov. pp. 41-46.

DiBernardo, A. and Lovell, C. W. (1982). "Compactive Prestress Effects in Clays", TRB Compaction Symposium, Washington, D.C. Jan. Scheduled for Publication.

Essigmann, M. F., Altschaeffl, A. G. and Lovell, C. W., "A Proposed Method for How Much Compaction to Specify", Transportation Research Record No. 690, TRB, 1978.

Liang, Y. and Lovell, C. W. "Predicting the Strength of Field Compacted Soil from Laboratory Tests", Special Publication 22, Colorado Geological Survey, Sept. 1982, pp. 186-201.

Liang, Y. and Lovell, C. W. "Strength of Field Compacted Clays", Canadian Geotechnical Journal, Vol. 20, No. 1, Feb. 1983, pp. 36-46.

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Lovell, C. W. (1979). "Prestress Established by Compaction". Discussion to the 7th European Conference on Soil Mechanics and Foundation Engineering, Proceedings, Vol. 4, 6.8, p. 223. Brighton, England.

Lovell, C. W. (1981). "Compaction Prestress Makes a Difference", Proceedings, 30th Annual Highway Geology Symposium, Portland, Oregon, Oct. 1981, pp. 287-307.

Lovell, C. W. and Johnson, J. M. (1981). "Shearing Behavior of Compacted Clay After Saturation", Laboratory Shear Strength of Soil, ASTM, STP 740, Sept. 1981. pp. 277-293.

Lovell, C. W. "Compacted Fills: Predicting Field Compacted Behavior from Laboratory Compacted Samples". Proceedings, 8th European Conference on Soil Mechanics and Foundation Engineering, Helsinki, Finaland, Vol. 1, May 1983, 4 pp.

Price, J. T., Altschaeffl, A. G. and Lovell, C. W., "Predicting Field Compacted Strength and Variability", Transportation Research Record No. 705, TRB, 1979.

Scott, J. C., Altschaeffl, A. G. and Lovell, C. W., "Soaked Strength of a Laboratory Compacted Clay", Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT11, November 1979.

Weitzel, D. W. and Lovell, C. W. (1980). "Prediction of Density and Strength for a Laboratory Compacted Clay", Transportation Research Record 754, Transportation Research Board, Washington, D.C., Nov., pp. 53-56.

APPENDIX F

Illustrations from Interim Reports

Page of Citation in Final Report	Illustration	Interim Report No.	Figure No. This Report
16	Figure 37	79-3	F1
16, 19R	Figure 38	79-3	F2
16	Figure 39	79-3	F3
19R	Equation 3-5	79-7	F4
19R	Figure 52	79-7	F5
19R	Figure 51	79-7	F6
19R	Equation 3-8	79-7	F7
20R	Figure 63	79-7	F8
20R	Equation 3-6	79-7	F9
20R	Figure 59	79-7	F10
22R	Equation 3-8	79-11	F11
22R	Table 3-6	79-11	F12
22R	Equation 3-9	79-11	F13
22R	Equation 3-11	79-11	F14
22R	Equation 3-15	79-11	F15
22R	Figure 3.30	79-11	F16
23R	Equation 3.18	79-11	F17
23R	Figure 3.31	79-11	F18
23R	Figure 3.32	79-11	F19

23R	Figure 3.33	79-11	F20
23R	Figure 3.34	79-11	F21
23R	Figure 3.35	79-11	F22
23R	Figure 3.37	79-11	F23
23R	Figure 3.38	79-11	F24
26R	Figure 32	81-14	F25
26R	Figure 33	81-14	F26
26R	Figure 34	81-14	F27
26R	Figure 35	81-14	F28
26R	Figure 36	81-14	F29
29	Table 3.22	82-1	F30
29	Equation 3-24	82-1	F31
29	Equation 3-25	82-1	F32
29	Equation 3-27	82-1	F33

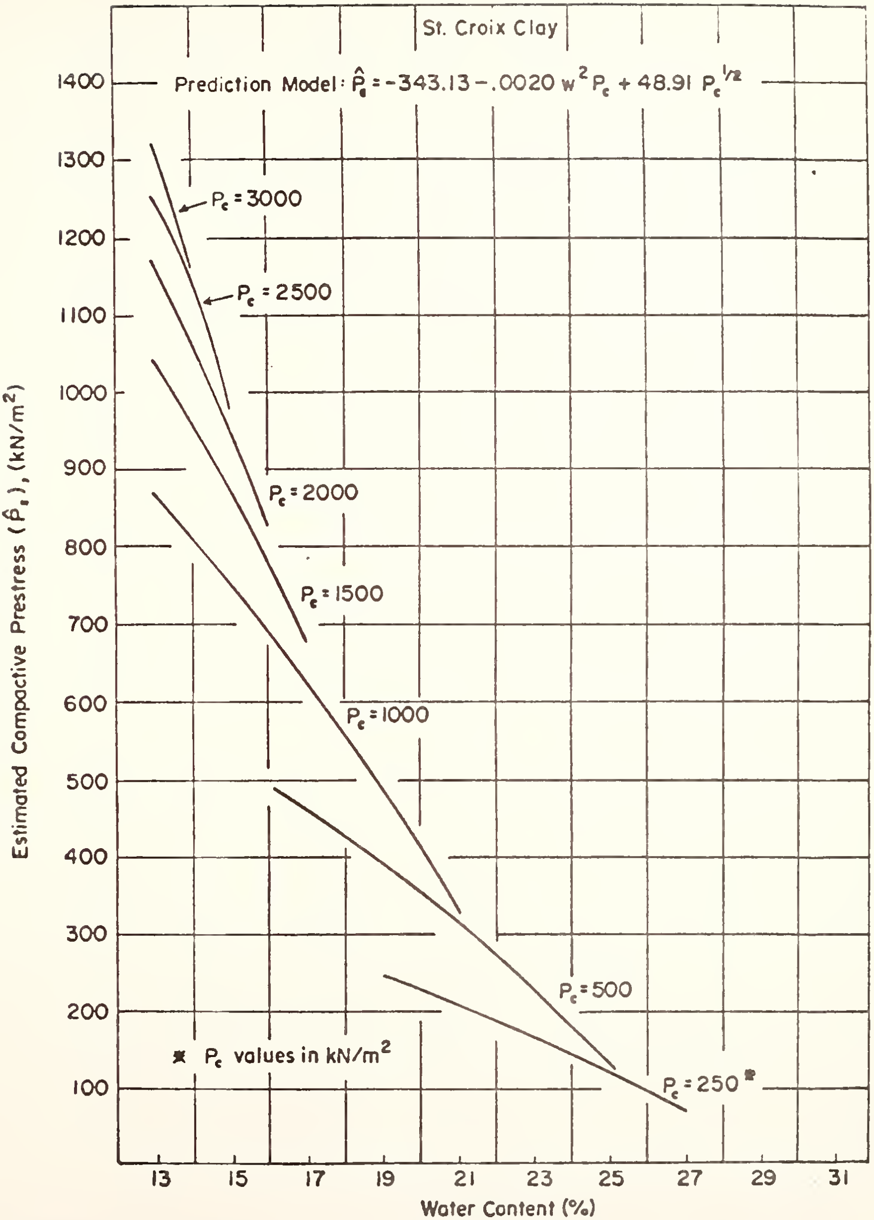


FIGURE 37 PREDICTION OF COMPACTIVE PRESTRESS

Figure F1.

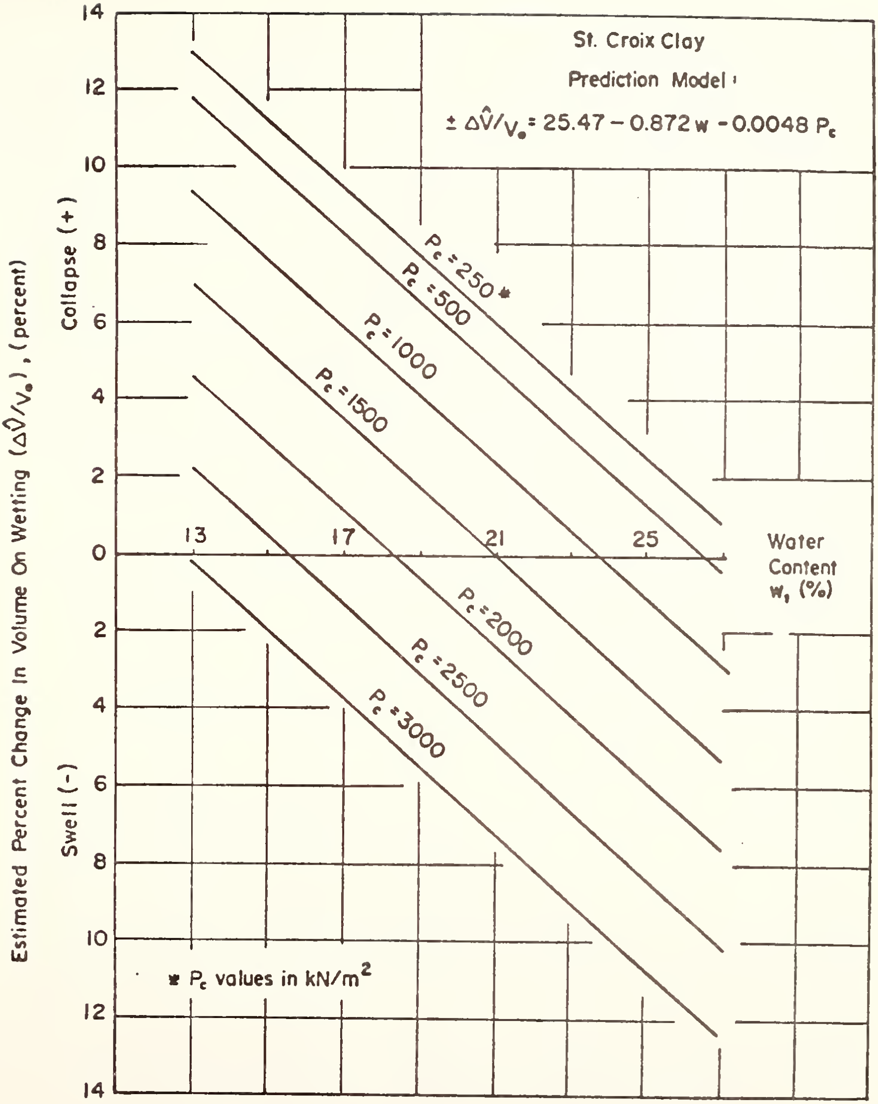


FIGURE 38 PREDICTION OF PERCENT VOLUME CHANGE ON WETTING

Figure F2.

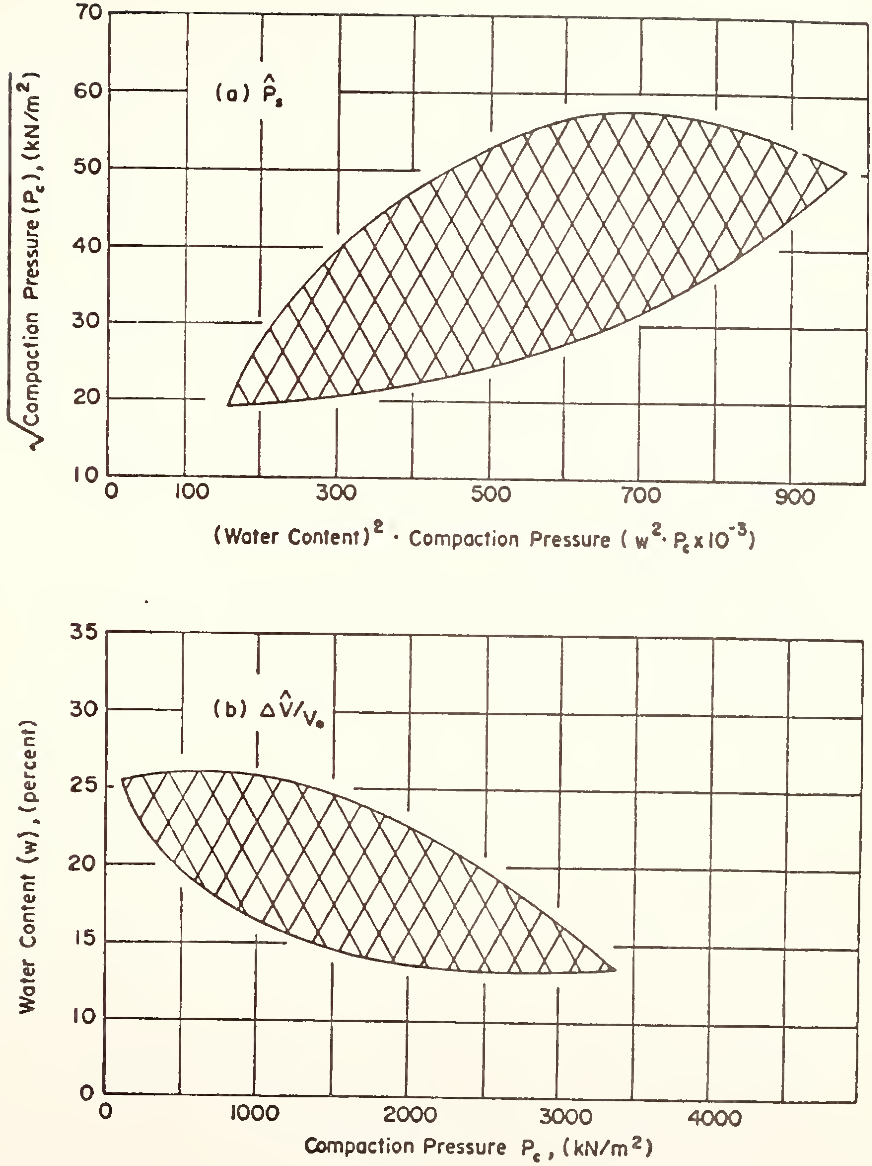


FIGURE 39 JOINT REGIONS OF OBSERVATIONS FOR PREDICTION MODELS

Figure F3.

$$\Delta\hat{V}/V_o\ (\%) = 28.48 - 0.0000136\rho_d^2 + 0.0077S_i\sqrt{\sigma_c'} \tag{3-5}$$

$\Delta\hat{V}/V_o(\%)$ = estimated value of percent volume change due to saturation and consolidation (%)

ρ_d = as-compacted dry density (kg/m^3)

S_i = initial degree of saturation (%)

σ_c' = isotropic consolidation pressure (kN/m^2)

Figure F4.

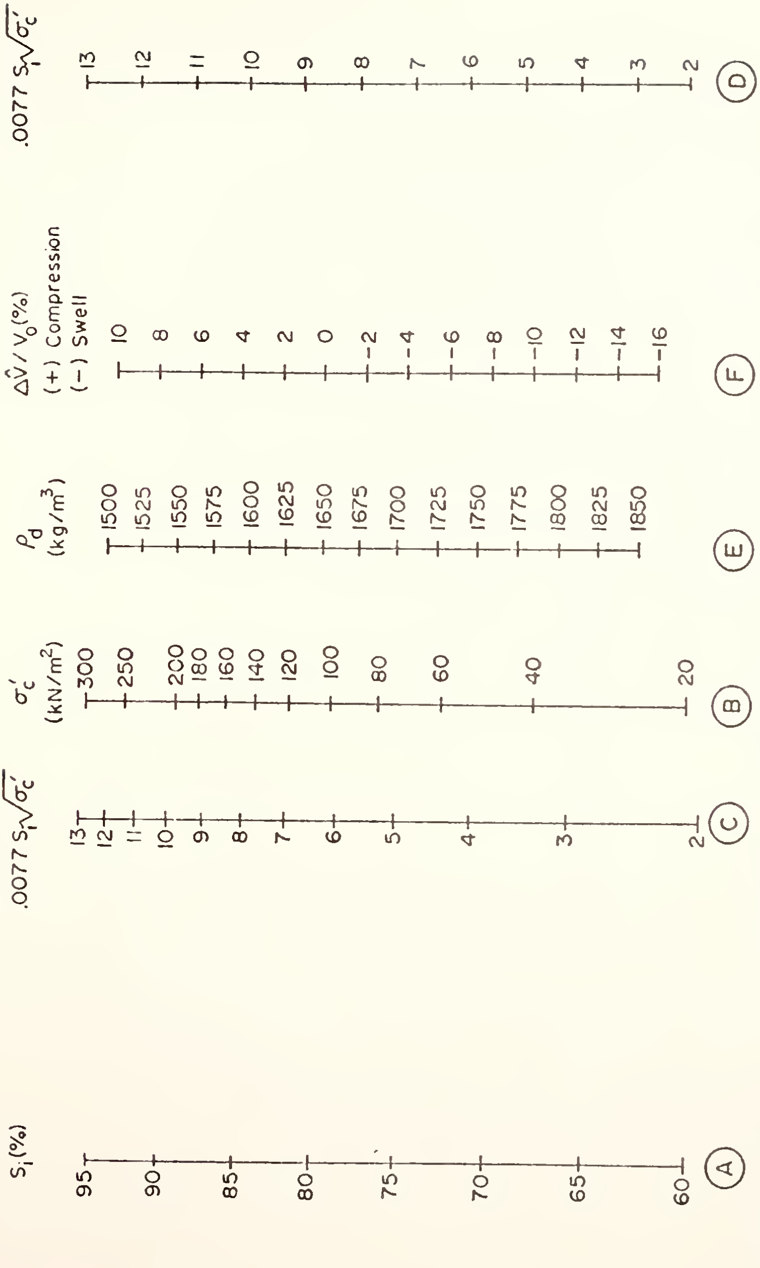


FIGURE 52 PREDICTION NOMOGRAPH FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION, $\Delta\hat{V}/V_o$(%)

Figure F5

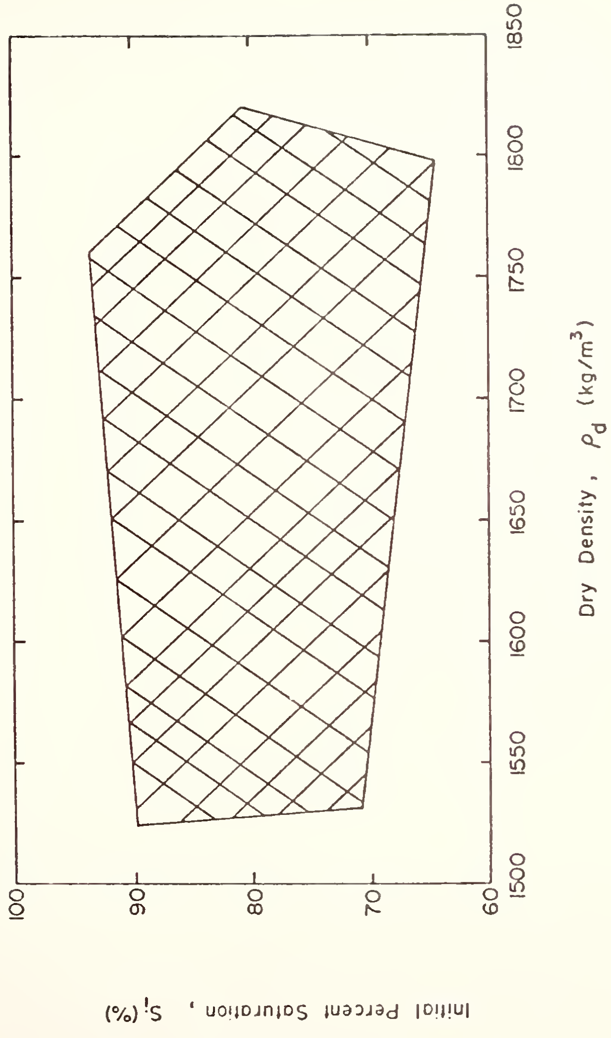


FIGURE 51 JOINT REGION OF INITIAL PERCENT SATURATION AND DRY DENSITY OBSERVATIONS

Figure F6.

$$\hat{c}' = 1.71 - 3.83\ w \log e_o \qquad (3\text{-}8)$$

$\hat{c}'$ = estimated value of the effective stress strength intercept (kN/m^2)

w = compaction moisture content (%)

e_o = initial void ratio

Figure F7.

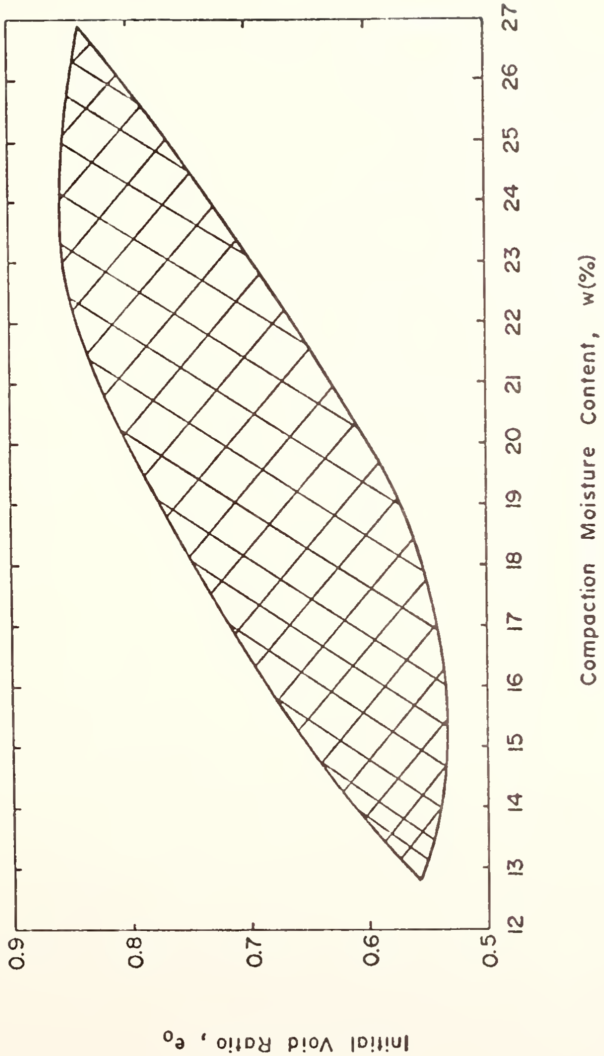


FIGURE 63 JOINT REGION OF INITIAL VOID RATIO AND MOISTURE CONTENT OBSERVATIONS

Figure F8.

$$\hat{A}_f = 1.79 - 0.00011\,\rho_d\sqrt{S_i} + 1.28\,\sigma'_c/\rho_d \qquad (3\text{-}6)$$

$\hat{A}_f$ = estimated value of Skempton's A parameter at failure

ρ_d = as-compacted dry density (kg/m^3)

S_i = initial degree of saturation (%)

σ'_c = isotropic consolidation pressure (kN/m^2)

Figure F9.

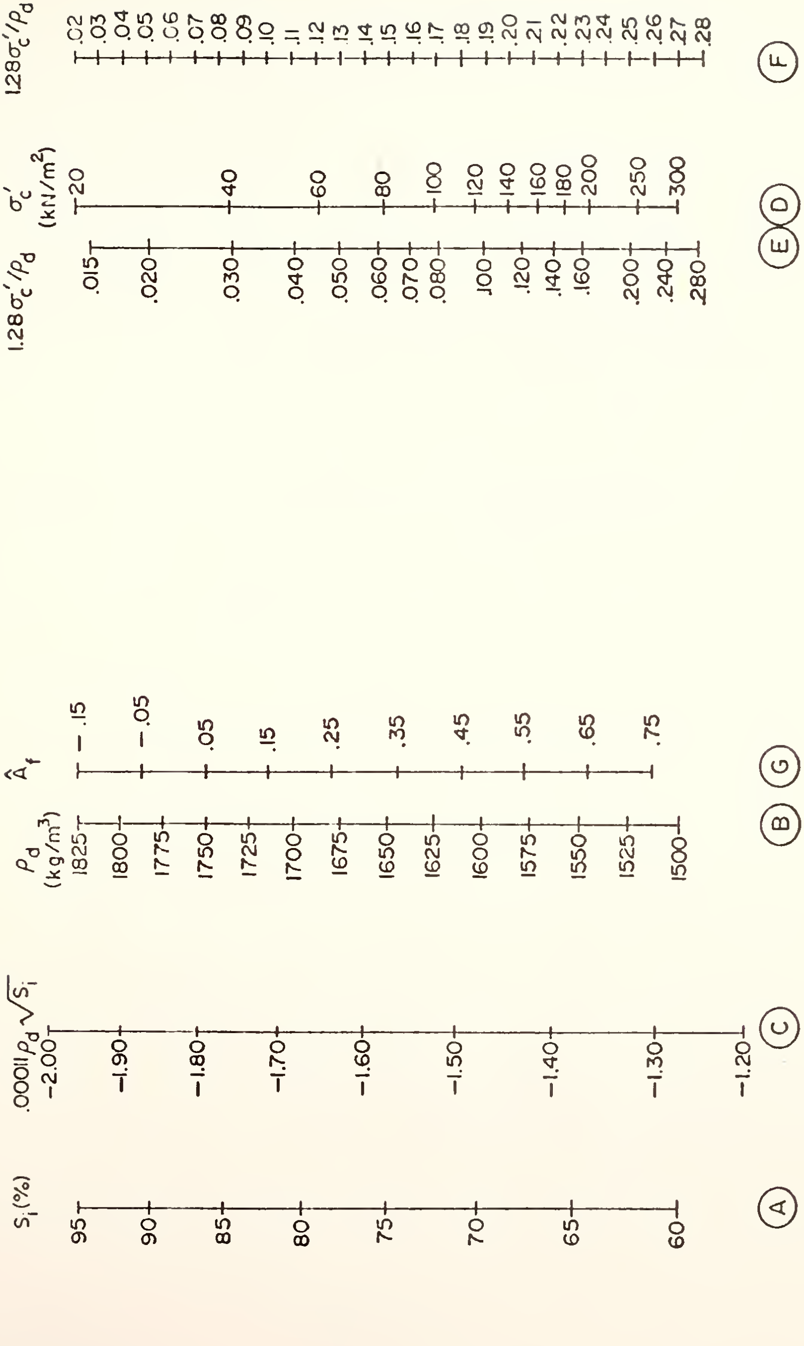


FIGURE 59 PREDICTION NOMOGRAPH FOR SKEMPTON'S A PARAMETER AT FAILURE, $\hat{A}_f$

Figure F10.

$$\hat{\rho}_d = 1338.3 + 1284.0\sqrt{W_R}/w + 0.32\, w^2\sqrt{W_R} \tag{3.8}$$

where: $\hat{\rho}_d$ = the estimated dry density, kg/m^3

W_R = average work ratio

w = water content, %

Figure F11.

Table 3.6 Differences Between Average Work at Different Nominal Energy-Saturation Levels and Average Work Ratios

(a) Water Content-Energy Level	Differences Between Average Work, kJ/m^3
L1 , S1	142
L1 , M1	1372
L2 , S2	224
L2 , M2	1443
L3 , S3	168
L3 , M3	1569
L4 , S4	97
L4 , M4	1403

(b) Energy Level	Average of Differences, kJ/m^3
L , S	158
L , M	1447

(c) Energy Level	Average Work Ratio (average of work for all L tests as denominator)
L	1.00
S	1.88
M	9.09

Figure F12

$$\hat{\rho}_d = 961.8 + 15564.6/w \qquad (3.9)$$

where:

$\hat{\rho}_d$ = predicted dry density, kg/m^3

w = water content, %

Figure F13.

$$\hat{q}_c = -1784.8 + 3.1\ \rho_d \sqrt{S_i}/w + 84.0\ (1 - S_i/100)\sqrt{\sigma_3} \qquad (3.11)$$

where:
$\hat{q}_c$ = estimated compressive strength, kN/m^2
ρ_d = dry density, kg/m^3
S_i = initial degree of saturation, %
w = water content, %
σ_3 = confining pressure, kN/m^2

Figure F14.

$$\log(\hat{q}_c) = 1.70/e_i \tag{3.15}$$

where; $\hat{q}_c$ = the estimated compressive strength, kN/m^2

e_i = initial void ratio

Figure F15.

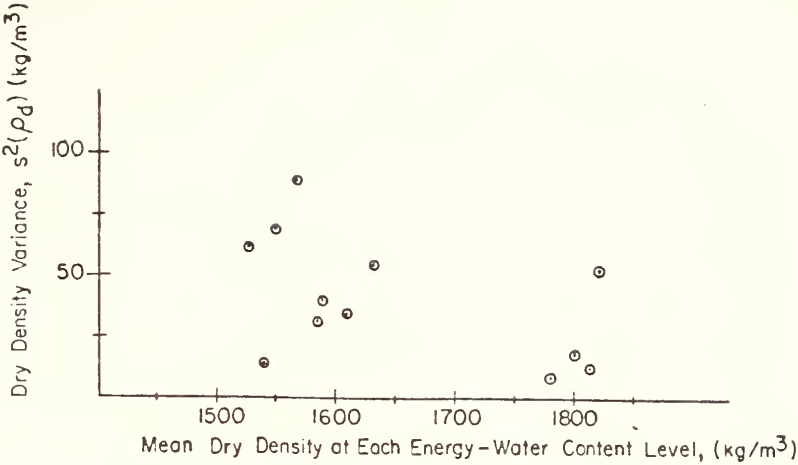


FIGURE 3.29 PLOT FOR VARIABILITY ANALYSIS OF DRY DENSITY

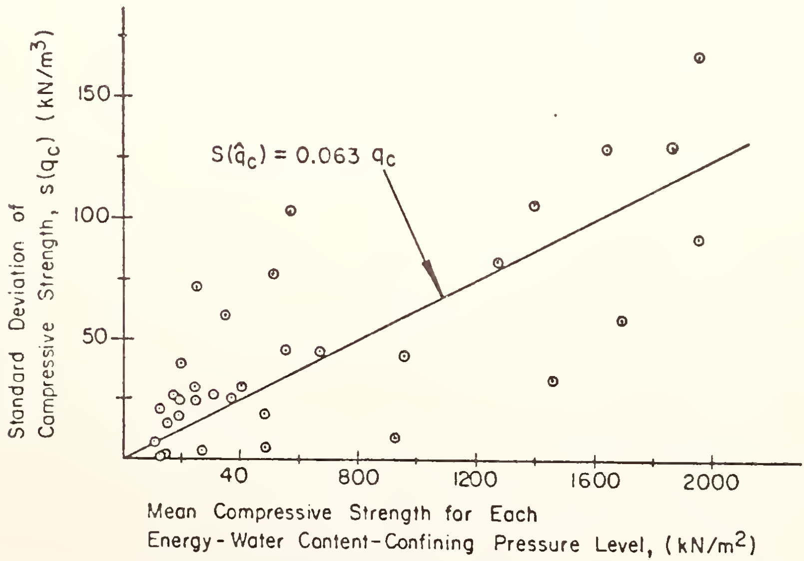


FIGURE 3.30 PLOT FOR VARIABILITY ANALYSIS OF COMPRESSIVE STRENGTH

Figure F16.

$$\hat{q}_{c_m} = q_c - V(\hat{q}_c) = q_c - 0.126\hat{q}_c \qquad (3.18)$$

where

$\hat{q}_{c_m}$ = minimum expected strength

q_c = expected mean strength

$V(\hat{q}_c)$ = twice the estimated standard deviation of compressive strength

and all units are kN/m^2.

Figure F17.

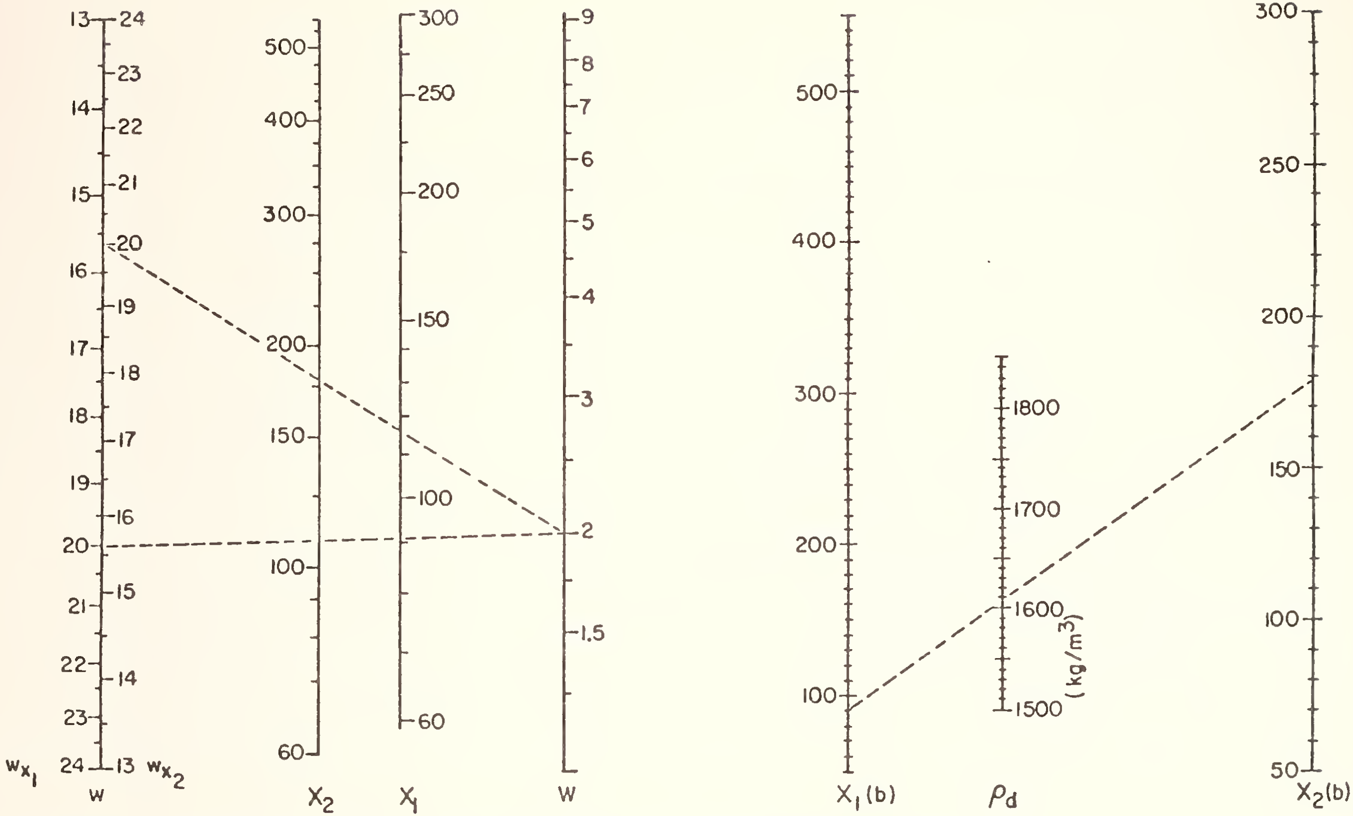


FIGURE 3.31 NOMOGRAPH FOR DRY DENSITY PREDICTION DRY-OF-OPTIMUM

1. See Figure 3.21 for acceptable values of W_R and w.
2. W_R for kneading compaction.

Figure F18.

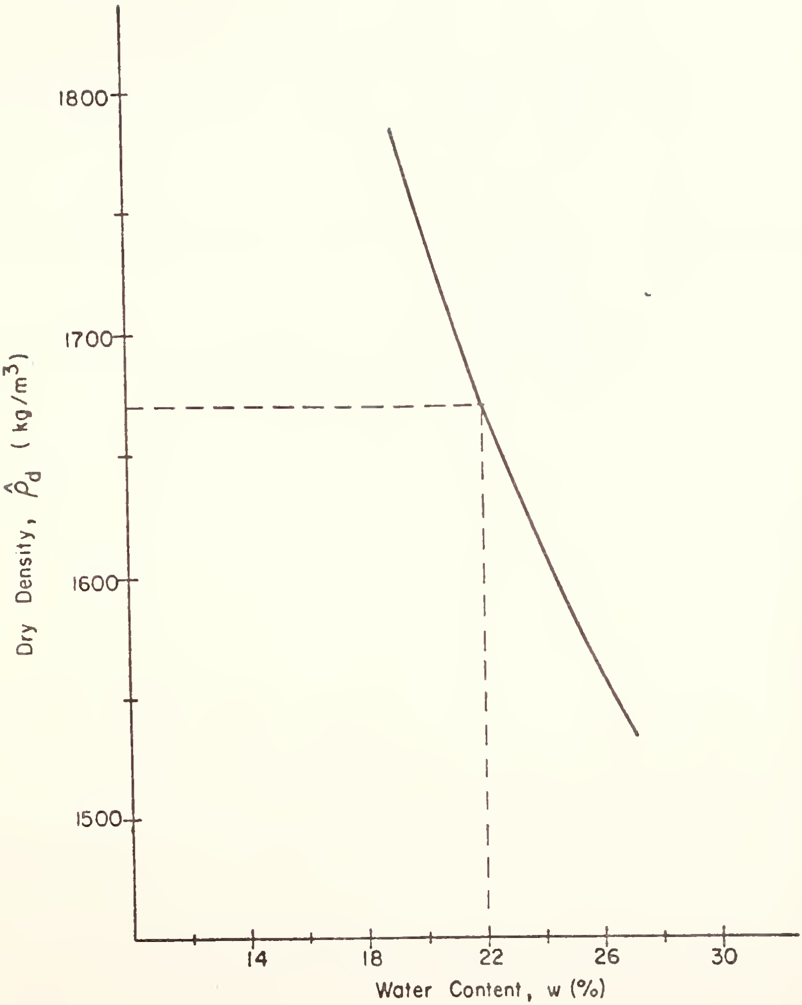


FIGURE 3.32 CHART FOR PREDICTION OF DRY DENSITY WET-OF-OPTIMUM

Figure F19.

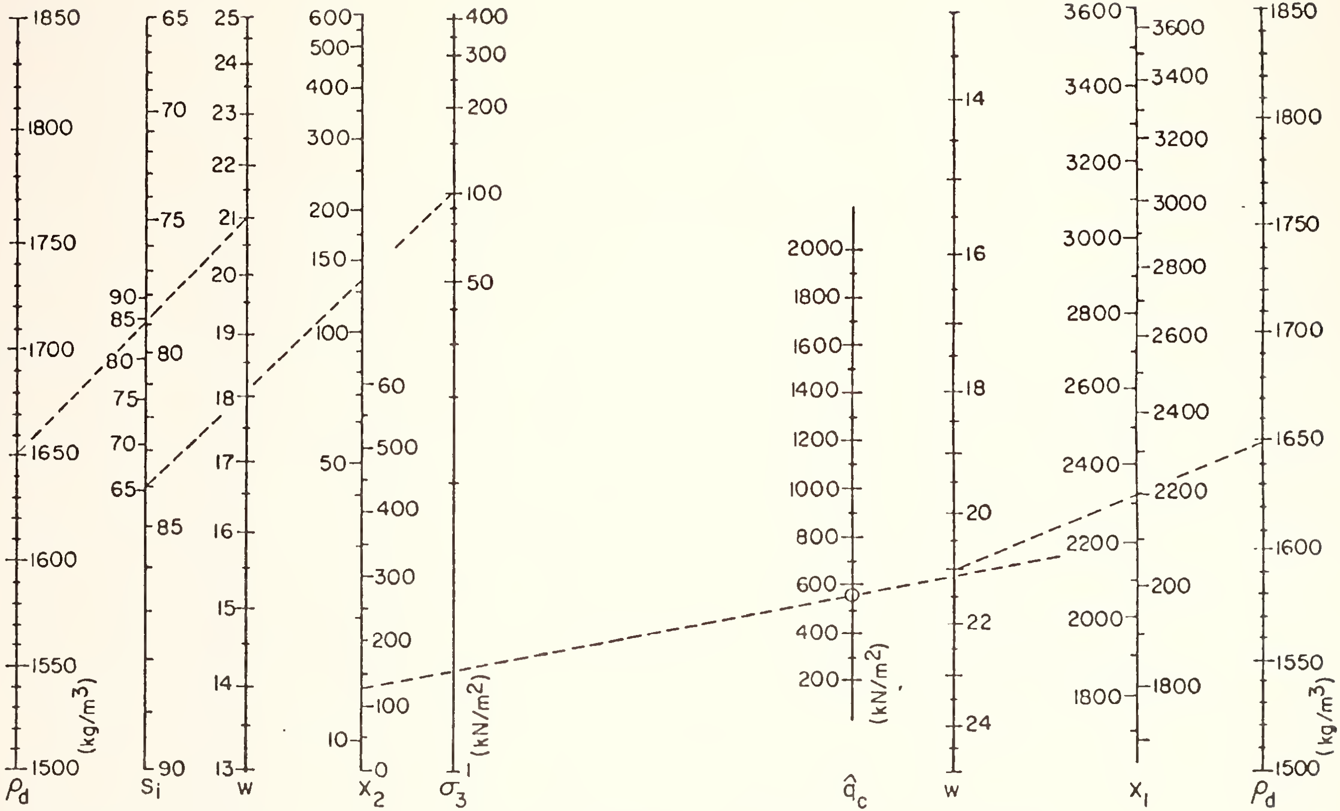


FIGURE 3.33 NOMOGRAPH FOR UNCONSOLIDATED-UNDRAINED STRENGTH PREDICTION FOR MOLDING WATER CONTENTS DRY-OF-OPTIMUM

See Figure 3.25 for acceptable values of ρ_d and w.

Figure F20.

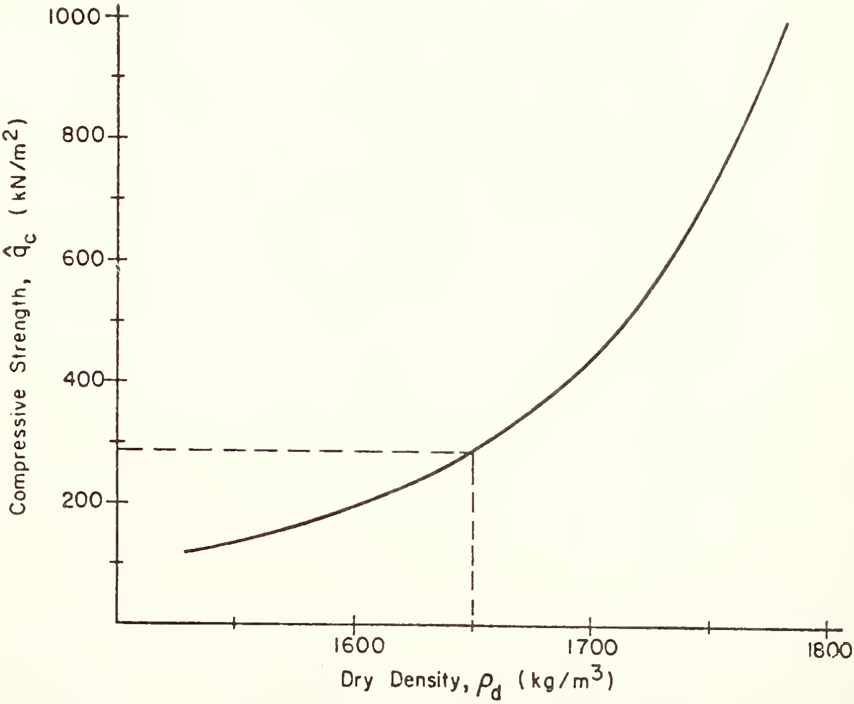


FIGURE 3.34 CHART FOR PREDICTION OF UNCONSOLIDATED - UNDRAINED STRENGTH FOR MOLDING WATER CONTENTS WET-OF-OPTIMUM.

Figure F21.

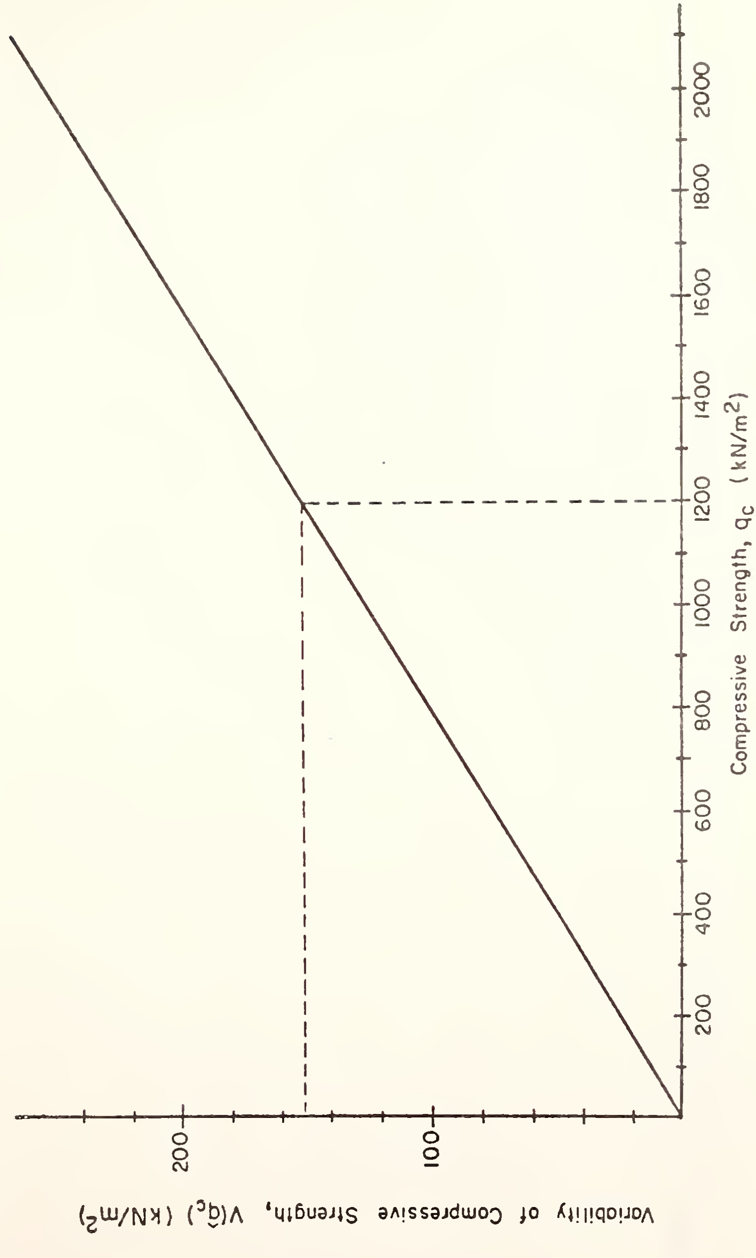


FIGURE 3.35 CHART FOR PREDICTION OF COMPRESSIVE STRENGTH VARIABILITY

Figure F22.

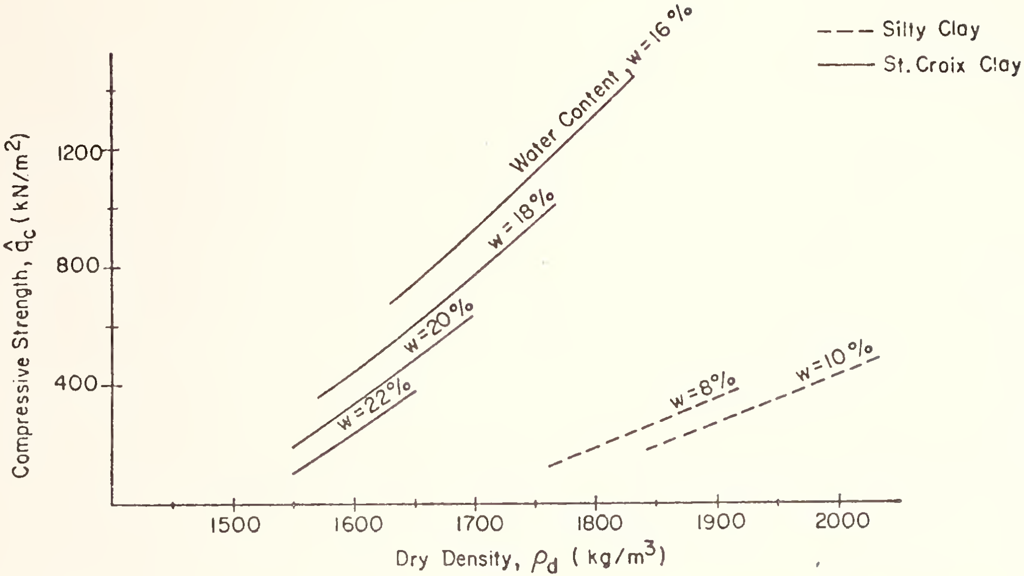


FIGURE 3.37 COMPARISON OF DRY-OF-OPTIMUM UNCONFINED COMPRESSIVE STRENGTH PREDICTION MODELS FOR SILTY CLAY AND ST. CROIX CLAY

Figure F23.

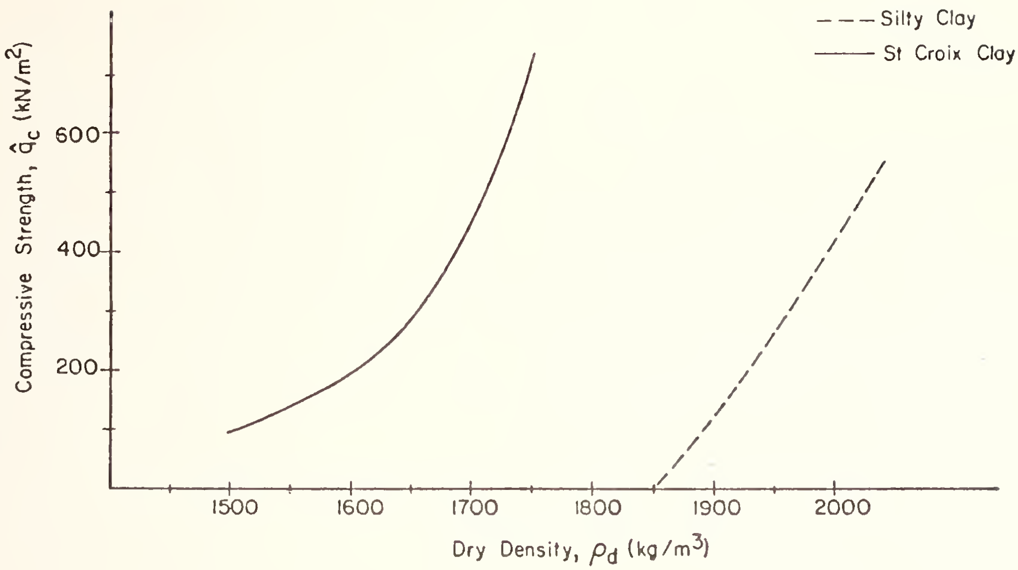


FIGURE 3.38 COMPARISON OF WET-OF-OPTIMUM COMPRESSIVE STRENGTH PREDICTION MODELS FOR SILTY CLAY AND ST. CROIX CLAY

Figure F24.

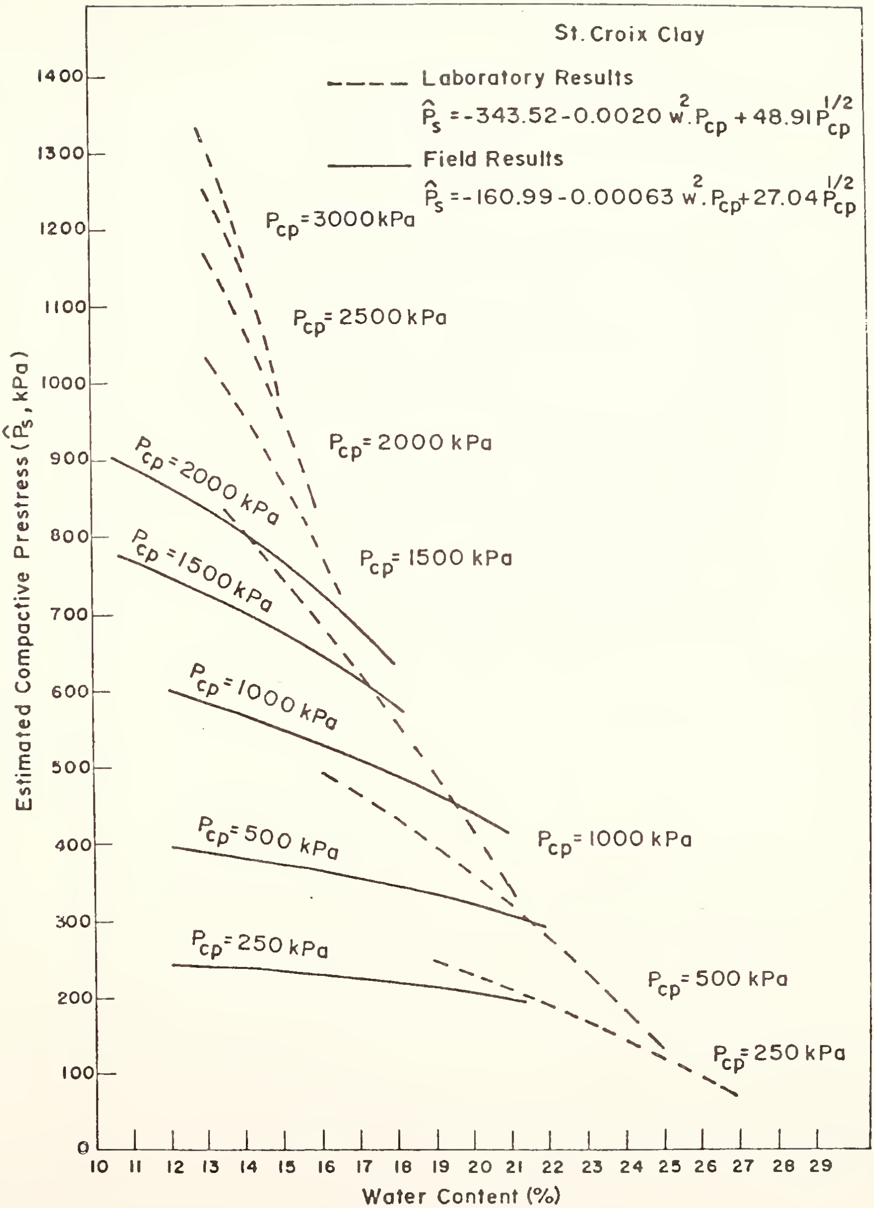


FIGURE 32 COMPACTIVE PRESTRESS MODELS

Figure F25.

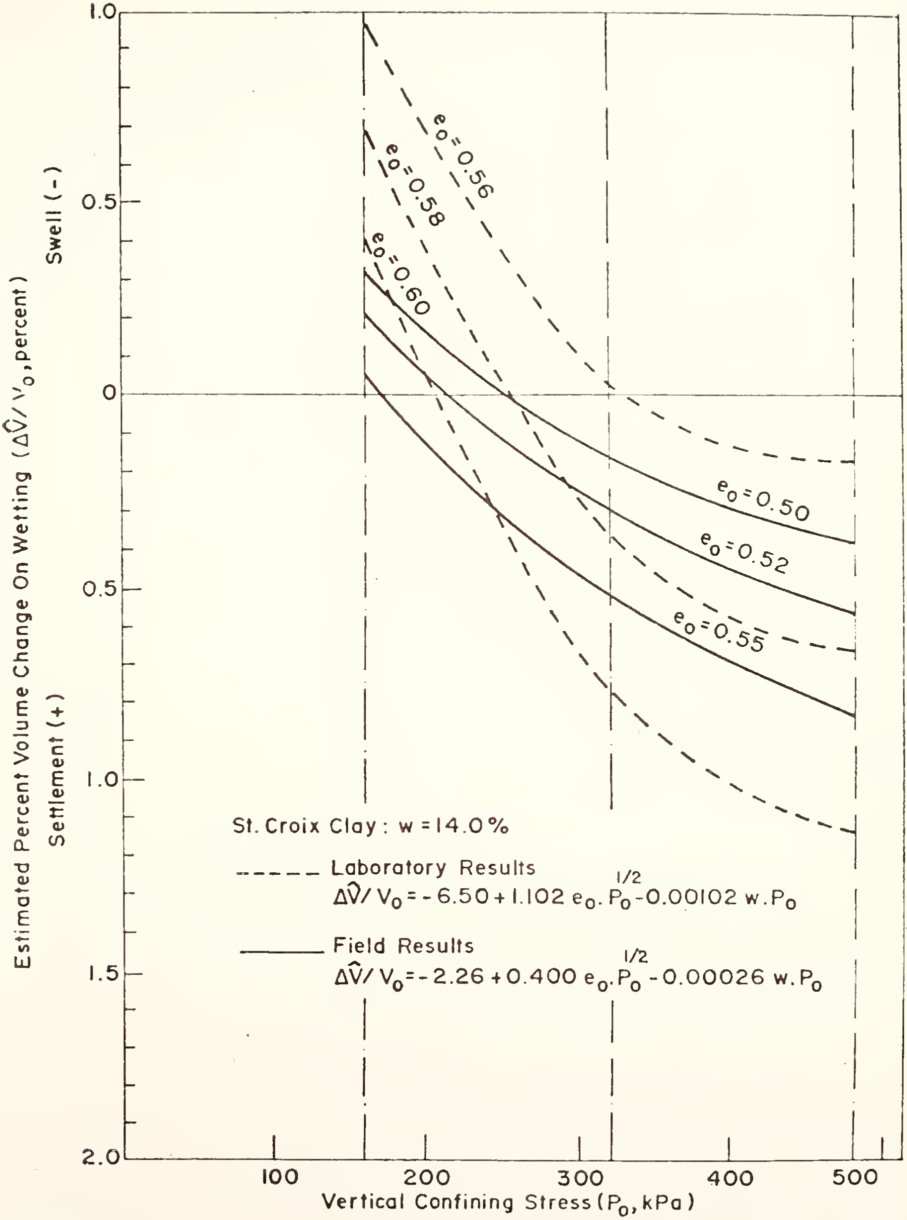


FIGURE 33 EFFECT OF VOID RATIO ON PERCENT VOLUME CHANGE MODEL(CONSTANT WATER CONTENT)

Figure F26.

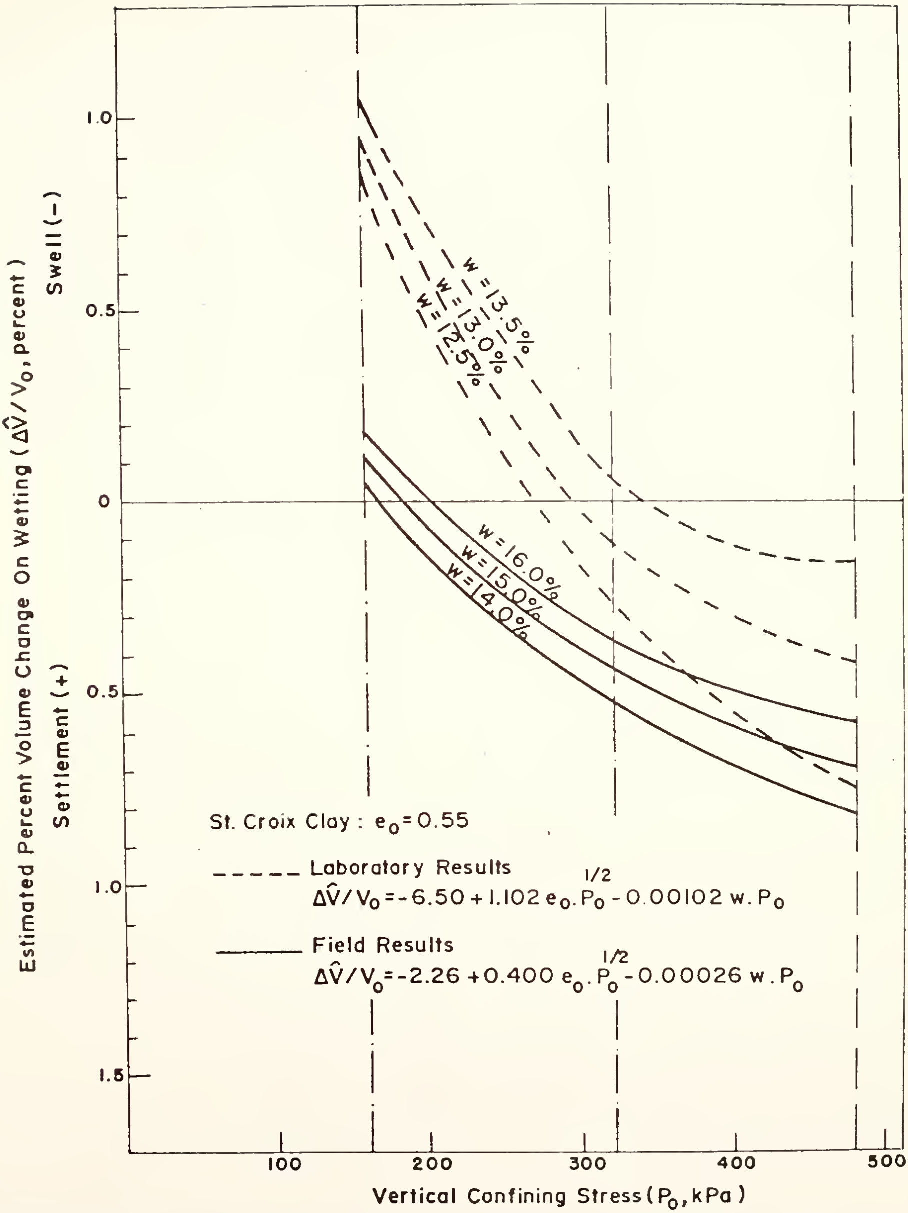


FIGURE 34 EFFECT OF WATER CONTENT ON PERCENT VOLUME CHANGE MODEL(CONSTANT VOID RATIO)

Figure F27.

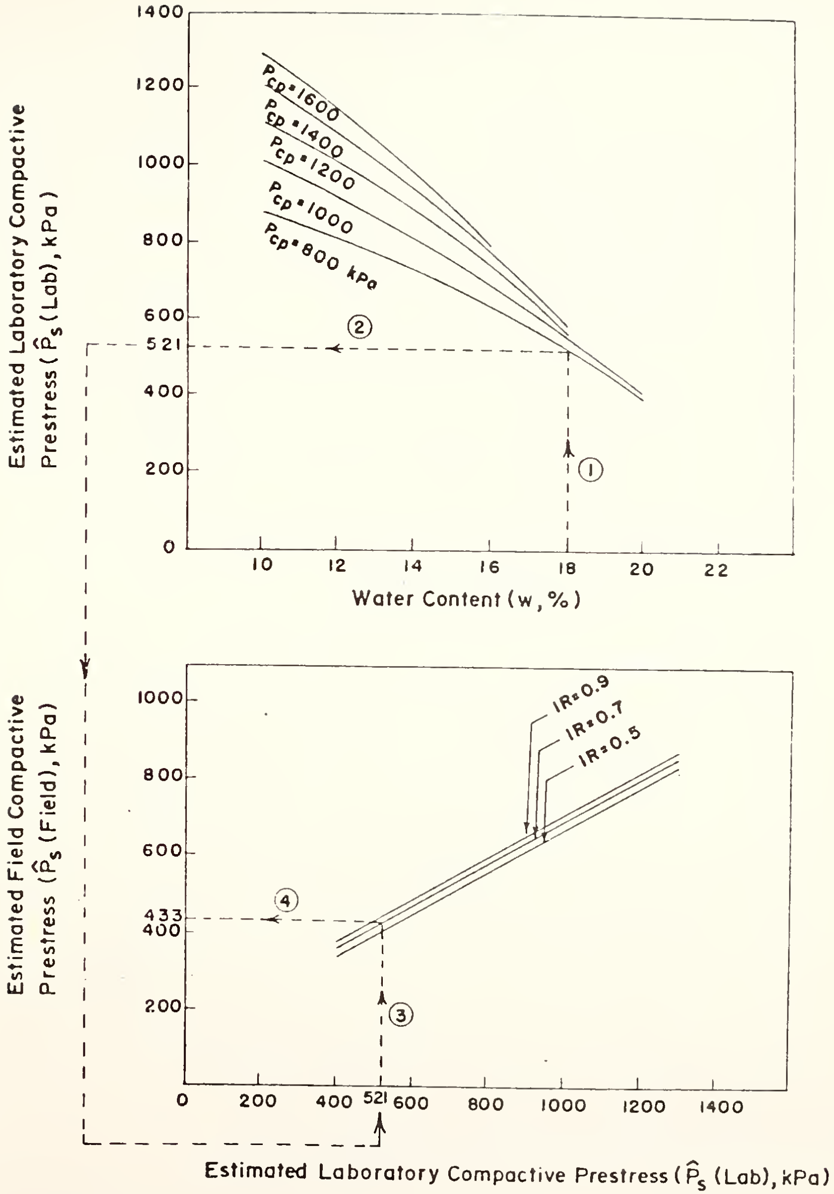
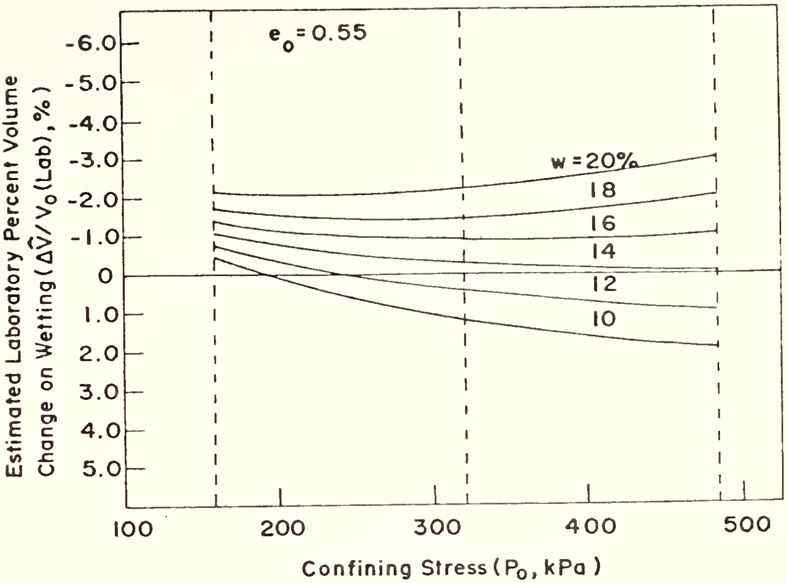


FIGURE 35 PREDICTION OF FIELD COMPACTIVE PRESTRESS FROM LABORATORY COMPACTED SAMPLES

Figure F28.



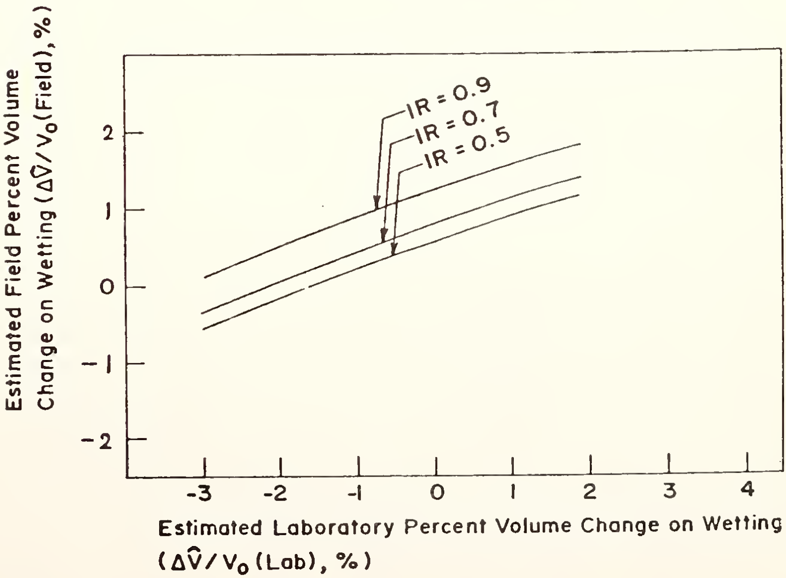


FIGURE 36 PREDICTION OF FIELD PERCENT VOLUME CHANGE ON WETTING FROM LABORATORY COMPACTED SAMPLES

Figure F29.

TABLE 3-22 Regression Results

Compactor Type	Dependent Variable	Regression Model	R^2
Both Types (Field)	Dry Density (all moisture)	$\hat{\rho}_d = 1929.68 + 211.6\sqrt{P_c}/w + 0.0016\sqrt{P_c} \cdot w^2 - 0.0096\, w \cdot P_c - 6816.83/w$	0.74
	Dry Density (wet-of-optimum)	$\hat{\rho}_d = 1273.05 + 8797.21\, w$	0.88
Rascal	As-Compacted Strength	$\hat{q}_c = -1310.75 - 117.4\, w + 0.23\, w^2 + 0.08\, \rho_d \cdot w + 122.77\,(1 - Si/100)\sqrt{\sigma_3} + 1.02\, \rho_d \sqrt{Si/w}$	0.76
Caterpillar		$\hat{q}'_c = -12546.07 + 1432.38\, w - 18.76\, w^2 - 0.39\, \rho_d\, w + 114.52\,(1 - Si/100)\sqrt{\sigma_3} + 5.73\, \rho_d \sqrt{Si/w}$	0.75
Both Types (Field)		$\hat{q}_c = -6980.05 + 636.21\, w - 8.3\, w^2 - 0.155\, \rho_d \cdot w + 112.1\,(1 - Si/100) \cdot \sqrt{\sigma_3} + 3.6\, \rho_d \cdot \sqrt{Si/w}$	0.72
Rascal	Percent Volume Change Due to Saturation and Consolidation	$\widehat{(\frac{\Delta V}{V})}\% = -4.31 + 2.41\, e_o \cdot \sqrt{\sigma'_c} - 0.168\sqrt{\hat{P}_s} - 0.286 \times 10^{-2} \cdot w \cdot \sigma'_c$	0.81
Caterpillar		$\widehat{(\frac{\Delta V}{V})}\% = 2.81 + 2.5 \cdot e_o \sqrt{\sigma'_c} - 0.52\sqrt{\hat{P}_s} - 0.25 \times 10^{-2} \cdot w \cdot \sigma'_c$	0.66
Both Types (Field)		$\widehat{(\frac{\Delta V}{V})}\% = -0.166 + 2.47\, e_o \sqrt{\sigma'_c} - 0.365\sqrt{\hat{P}_s} - 0.00263 \cdot w \cdot \sigma'_c$	0.72

Figure F30.

TABLE 3-22 (Continued)

Compactor Type	Dependent Variable	Regression Model	R^2
Rascal	Skempton's A Parameter at Failure	$\hat{A}_f = 1.92 - 0.55/e_o - 0.368 \times 10^{-4}\ \rho_d \cdot \sqrt{S_i} - 0.365 \log(OCR)$	0.60
Caterpillar		$\hat{A}_f = 2.42 - 0.95/e_o - 0.192 \times 10^{-4}\ \rho_d \cdot \sqrt{S_i} - 0.416 \log(OCR)$	0.72
Both Types (Field)		$\hat{A}_f = 2.05 - 0.73/e_o - 0.232 \times 10^{-4}\ \rho_d \sqrt{S_i} - 0.382 \log(OCR)$	0.63
Both Types (Field)	Effective Stress Strength Intercept	$\hat{c}' = -102.79 + 11.208\ w + 14.55\ w \cdot \log e_o$	0.97
Both Types (Field)	Effective Stress Strength Angle	$\hat{\phi}' = 47.56 - 2.112\ w - 2.625\ w \cdot \log e_o$	0.89

Figure F30 (con't)

$$\hat{q}_c \text{ (field)} = 174.92 + 0.916\ \hat{q}_c \text{ (lab)} \qquad (3\text{-}24)$$

where $\hat{q}_c$ = expected value of as-compacted compressive strength, kN/m^2.

Figure F31.

$$\hat{\Delta V}/V_o \text{ (field)} = 1.886 + 0.495 \quad \hat{\Delta V}/V_o \text{ (lab)} \tag{3-25}$$

where $\hat{\Delta V}/V_o$ = expected value of volumetric strain in %

Figure F32.

$$\hat{A}_f \text{ (field)} = 0.223 + 0.66\ \hat{A}_f \text{ (lab)}$$

where $\hat{A}_f$ = expected value of Skempton's A parameter at failure

Figure F33.

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